DOCUMENT RESUME

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TITLE	Military Curriculum Materials for Vocational and Technical Education. Construction Surveying, 3-27.
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IDENTIFIERS	Military Curriculum Project; *Surveying (Construction)

ABSTRACT

This short course on construction surveying was developed from military curriculum materials for use in technical and vocational education programs. Students completing the course should be able to perform engineering surveys related to area and route surveying (knowledge of basic survey techniques is a prerequisite). The course is divided into two units. Unit 1 contains a 1-hour lesson introducing the course and covering safety aspects of the job. Unit 2 contains 13 lessons totaling 29 hours of classroom and 66 hours of practical instruction covering the following topics: construction surveying; horizontal control; vertical control; topographic survey; road surveying; road surveys; horizontal curves; vertical curves; grade and earth-work computations; engineering transit adjustment; level adjustment; utility surveying; and building layout. The course contains both student and teacher materials. Printed teacher materials include an introduction to the course; outline of instruction; outline of training objectives; lists of texts, references, tools, equipment, materials, training aids and devices, and training aids equipment; a master schedule; and instructor guides. Instructor guides include lesson plans and activities for instructor and students. Information sheets, job sheets, and practical test sheets are provided for students. In addition, appropriate chapters of two Navy manuals to be used in the course are provided, and transparency sets and references are suggested. (KC)

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MILITARY CURRICULUM MATERIALS

The military-developed curriculum materials in this course package were selected by the National Center for Research in Vocational Education Military Curriculum Project for dissemination to the six regional Curriculum Coordination Centers and other instructional materials agencies. The purpose of disseminating these courses was to make curriculum materials developed by the military more accessible to vocational educators in the civilian setting.

The course materials were acquired, evaluated by project staff and practitioners in the field, and prepared for dissemination. Materials which were specific to the military were deleted, copyrighted materials were either omitted or approval for their use was obtained. These course packages contain curriculum resource materials which can be adapted to support vocational instruction and curriculum development.

The National Center Mission Statement

The National Center for Research in Vocational Education's mission is to increase the ability of diverse agencies, institutions, and organizations to solve educational problems relating to individual career planning, preparation, and progression. The National Center fulfills its mission by:

- Generating knowledge through research
- Developing educational programs and products
- Evaluating individual program needs and outcomes
- Installing educational programs and products
- Operating information systems and services
- Conducting leadership development and training programs

FOR FURTHER INFORMATION ABOUT
Military Curriculum Materials
WRITE OR CALL
Program Information Office
The National Center for Research in Vocational Education
The Ohio State University
1960 Kenny Road, Columbus, Ohio 43210
Telephone: 614/486-3655 or Toll Free 800/848-4815 within the continental U.S. (except Ohio)

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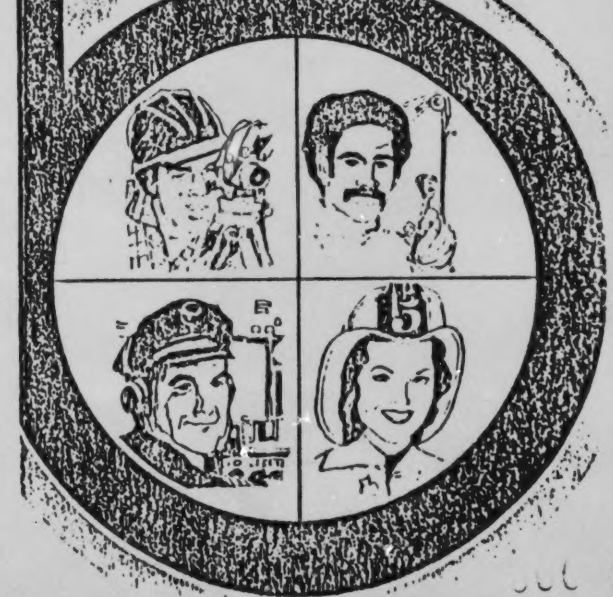
Military Curriculum Materials for Vocational and Technical Education

Information and Field Services Division

The National Center for Research in Vocational Education



Military Curriculum Materials Dissemination Is . . .

an activity to increase the accessibility of military-developed curriculum materials to vocational and technical educators.

This project, funded by the U.S. Office of Education, includes the identification and acquisition of curriculum materials in print form from the Coast Guard, Air Force, Army, Marine Corps and Navy.

Access to military curriculum materials is provided through a "Joint Memorandum of Understanding" between the U.S. Office of Education and the Department of Defense.

The acquired materials are reviewed by staff and subject matter specialists, and courses deemed applicable to vocational and technical education are selected for dissemination.

The National Center for Research in Vocational Education is the U.S. Office of Education's designated representative to acquire the materials and conduct the project activities.

Project Staff:

Wesley E. Budke, Ph.D., Director
National Center Clearinghouse

Shirley A. Chase, Ph.D.
Project Director

What Materials Are Available?

One hundred twenty courses on microfiche (thirteen in paper form) and descriptions of each have been provided to the vocational Curriculum Coordination Centers and other instructional materials agencies for dissemination.

Course materials include programmed instruction, curriculum outlines, instructor guides, student workbooks and technical manuals.

The 120 courses represent the following sixteen vocational subject areas:

- Agriculture
- Aviation
- Building & Construction Trades
- Clerical Occupations
- Communications
- Drafting
- Electronics
- Engine Mechanics
- Food Service
- Health
- Heating & Air Conditioning
- Machine Shop
- Management & Supervision
- Meteorology & Navigation
- Photography
- Public Service

The number of courses and the subject areas represented will expand as additional materials with application to vocational and technical education are identified and selected for dissemination.

How Can These Materials Be Obtained?

Contact the Curriculum Coordination Center in your region for information on obtaining materials (e.g., availability and cost). They will respond to your request directly or refer you to an instructional materials agency closer to you.

CURRICULUM COORDINATION CENTERS

EAST CENTRAL
Rebecca S. Douglass
Director
100 North First Street
Springfield, IL 62777
217/782-0759

MIDWEST
Robert Patton
Director
1515 West Sixth Ave.
Stillwater, OK 74704
405/377-2000

NORTHEAST
Joseph F. Kelly, Ph.D.
Director
225 West State Street
Trenton, NJ 08625
609/292-6562

NORTHWEST
William Daniels
Director
Building 17
Airdustrial Park
Olympia, WA 98504
206/753-0879

SOUTHEAST
James F. Shill, Ph.D.
Director
Mississippi State University
Drawer DX
Mississippi State, MS 39762
601/325-2510

WESTERN
Lawrence F. H. Zane, Ph.D.
Director
1776 University Ave.
Honolulu, HI 96822
808/948-7834

ENGINEERING AID SCHOOL, CONSTRUCTION SURVEYING

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Engineering Aid 3 & 2

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Chapter 15 - Topographic Surveying and Mapping Page 317

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Engineering Aid 1 & C

Chapter 6 - Construction and Land Surveys Page 374

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Chapter 8 - Horizontal and Vertical Curves Page 436

ENGINEERING AND SCHOOL, CONSTRUCTION SURVEYING

Classroom Course

Developed by:
United States Navy

Development and Review Dates
January 1976

D.O.T. No.:
005.081

Occupational Area:
Building and Construction

Target Audiences:
Grades 11-adult

Print Pages:
470

Cost:
$9.50

Availability:
Military Curriculum Project, The Center for Vocational Education, 1960 Kenny Rd., Columbus, OH 43210

Contents:	Type of Materials:	Lesson Plans:	Programmed Text:	Student Workbook:	Handouts:	Text Materials:	Audio-Visuals:	Instructional Design:	Performance Objectives:	Tests:	Review Exercises:	Additional Materials Required:	Type of Instruction:	Group Instruction:	Individualized:
Unit 1.1 **Introduction and Safety**		●							●	●		★		●	
Unit 1.2 **Construction Surveying**		●			●	●	★		●	●		★		●	●

★ Materials are recommended but not provided.

Expires July 1, 1978

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Course Description:

Students completing this short course will be able to perform engineering surveys related to area and route surveying. Knowledge of basic survey techniques is a prerequisite for this course.

The course is divided into two units. Unit 1.1—**Introduction and Safety** contains a one hour lesson introducing the course and covering safety aspects of the job. Unit 2—**Construction Surveying** contains thirteen lessons totaling 29 hours of classroom and 66 hours of practical instruction. The lesson titles and hours follow:

1.2.1	Construction Surveying (3 hours classroom)
1.2.2	Horizontal Control (3 hours classroom, 4 hours practical)
1.2.3	Vertical Control (1 hour classroom, 6 hours practical)
1.2.4	Topographic Survey (3 hours classroom, 7 hours practical)
1.2.5	Road Surveying (2 hours classroom, 7 hours practical)
1.2.6	Road Surveys (2 hours classroom, 4 hours practical)
1.2.7	Horizontal Curves (3 hours classroom, 9 hours practical)
1.2.8	Vertical Curves (2 hours classroom, 5 hours practical)
1.2.9	Grade and Earth Work Computations (2 hours classroom, 5 hours practical)
1.2.10	Engineering Transit Adjustment (2 hours classroom, 3 hours practical)
1.2.11	Level Adjustment (2 hours classroom, 3 hours practical)
1.2.12	Utility Surveying (2 hours classroom, 5 hours practical)
1.2.13	Building Layout (2 hours classroom, 8 hours practical)

This course contains both student and teacher materials. Printed teacher materials include an introduction to the course; outline of instruction; outline of training objectives; lists of texts, references, tools, equipment, materials, training aids and devices, and training aids equipment; a master schedule; and the instructor guides. The instructor guides include lesson plans, and activities for instructor and students. Information sheets, job sheets, and practical test sheets are provided for the students.

The texts used are two Navy training manuals, *Engineering Aid 3 & 2, NAVPERS 10634-B* and *Engineering Aid 1 & C, NAVEDTRA 10635-B*. The appropriate chapters are provided. In addition three military manuals and seven commercial books are recommended as references. Two transparency sets are also suggested but are not provided.

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3-27

SEABEES

SPECIAL CONSTRUCTION BATTALION TRAINING

ENGINEERING AID SCHOOL

410.2 CONSTRUCTION SURVEYING

JANUARY 1976

DEPARTMENT OF THE NAVY
NAVAL CONSTRUCTION TRAINING CENTER

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TITLE PAGE

TITLE: CONSTRUCTION SURVEYING

COURSE: Special Construction Battalion Training (SCBT) 410.2

COURSE LENGTH: 3 Weeks

CONTACT TIME: 96 hours

TAUGHT AT: Naval Construction Training Center,
Port Hueneme, Ca. 93043

Naval Construction Training Center
Port Hueneme, Ca 93043

CLASS CAPACITY:	Normal:	10
	Minimum:	06
	Maximum:	12

INSTRUCTOR REQUIREMENT PER CLASS: Class: 12/1
Pract:

COURSE CURRICULUM MODEL MANAGER: Naval Construction Training Center
Port Hueneme, Ca 93043

CURRICULUM CONTROL: Chief of Naval Technical Training

QUOTA MANAGEMENT AUTHORITY: School at which taught.

QUOTA CONTROL: School at which taught.

APPROVAL/IMPLEMENTATION DATE: When approved by the Chief of Naval Technical Training.

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HOW TO USE INSTRUCTOR GUIDES

Instructor Guides are provided for each topic and include supporting instructional materials and aids identified by the topic number and preceded by a letter code designation. The letter code key is as follows:

AS - Assignment Sheet
JS - Job Sheet
IS - Information Sheet
CN - Class Notes
OS - Operation Sheet
T - Test
FT - Final Test
TR - Transparencies
DS - Diagram Sheet
PS - Problem Sheet
PT - Pretest
PE - Performance Evaluation
WS - Work Sheet
G - General (give a definition of item)

A complete listing of all supporting materials and aids is documented with full descriptive titles in Annex.

The instructor guides are intended to be used as master lesson plans subject to personalization by the individual instructor. In all cases, it is expected that the instructor will study the references in preparation for annotating the guide. It is also expected that each instructor will develop an appropriate introduction for each topic that will (1) create interest, (2) show the value of the topic to the student, (3) relate the topic to previous and future topics in the course, and (4) communicate the learning objectives to the student. Well prepared introduction will then provide the important motivational conditioning to establish readiness and effect for learning appropriate to each topic.

The first page of each instructor guide contains the following functional information:

1. Topic of lesson.
2. Time in periods.
3. References.
4. Instructional Materials.
5. Instruction Aids.
6. Objectives.
7. Topic criterion test (as applicable).
8. Homework assignment (when applicable).
9. Tools and materials.

The pages following page 1 of each instructor guide provide in a three-column format the teaching/learning procedures for conducting the lesson. The left-hand column includes the outline of instructional content required by the objectives; the center column includes recommended instructor activities or methodology; the right-hand column contains recommended student learning activities.

While the methodology and student learning activities documented in each instructor guide have been tested and proven to be effective for the lead school, those schools implementing this curriculum are encouraged to exercise creativity in designing learning exercises and conceiving methods and techniques to meet course objectives.

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COURSE DATA PAGE

COURSE MISSION: To train selected Engineering Aids in the knowledge and technical skills defined by the Personnel Readiness and Capability Program for Engineering Aids 410.2.

PERSONNEL AND RATING ELIGIBLE: E-3 thru E-6.

OBLIGATED SERVICE: N/A

NOBC/NEC.: N/A

PHYSICAL REQUIREMENTS: N/A

SECURITY CLEARANCE REQUIRED: N/A

PREREQUISITES TRAINING AND/OR BASIC BATTERY TEST SCORE REQUIRED: Special Construction Battalion Training Course 410.1

RELATED TRAINING: None

FOLLOW-UP TRAINING : None

Performance will be evaluated on a go/no go basis.

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OUTLINE OF INSTRUCTION

PHASE I

TOPIC	Unit 1.1	CLASS	PRACT	TOTAL	PAGE
	Introduction and Safety				
1.1.1	Introduction and Safety	1	0	1	3
	Unit 2				
	Construction Surveying				
1.2.1	Construction Surveying	3	0	3	3
1.2.2	Horizontal Control	3	4	7	3
1.2.3	Vertical Control	1	6	7	4
1.2.4	Topographic Survey	3	7	10	4
1.2.5	Road Surveying	2	7	9	4
1.2.6	Road Surveys	2	4	6	4
1.2.7	Horizontal Curves	3	9	12	4
1.2.8	Vertical Curves	2	5	7	5
1.2.9	Grade and Earth Work Computations	2	5	7	5
1.2.10	Engineering Transit Adjustment	2	3	5	5
1.2.11	Level Adjustment	2	3	5	5
1.2.12	Utility Surveying	2	5	7	6
1.2.13	Building Layout	2	8	10	6
		29	66	95	

* Total Periods Classroom: 30
* Total Periods Practical: 66
Total Hours for Course: 96
Total Weeks for Course: 3 weeks.

* All periods represent 60 minutes of actual instruction.

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OUTLINE OF TRAINING OBJECTIVES

Unit 1.1 Introduction Contact Hours: 1

Terminal Objective: Upon completion of this unit the student will have registered for the course, received text books, and complied with NAVCONSTRACEN and CBC Regulations governing the reporting and fighting of fires which pertain to him as a SCBT student.

Topic 1.1.1 Introduction and Safety Contact Hours: 1

Enabling Objective: Upon completion of this topic the student will be able to answer orally specific questions pertaining to the mission, regulations and organization of the command, and the method of reporting and fighting fires as established by NAVCONSTRACEN and CBC regulations.

Unit 2. Construction Surveying Contact Hours: 95

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area abd route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Topic 1.2.1 Construction Surveying Contact Hours: 3

Enabling Objectives: Upon completion of this topic each student will identify the types of construction surveys, basic surveying procedures and safety practices, be issued surveying equipment on a party basis. In order to perform these requirements the student will use the student texts, Naval Mobile Construction Battalion Table of Allowance Kit 0045A/80010, Surveyors Equipment and instructor guidance. The minimum student requirements will be evaluated on his performance in the field for basic surveying procedures and safety practices while performing construction surveys required by the remaining topics of this unit and 100% percent accuracy when inventorying party kits.

Topic 1.2.2 Horizontal Control Contact Hours: 7

Enabling Objectives: Upon completion of this topic each student will while working as a member of a survey party and performing all duties of instrument man, rodman, notekeeper and chainman; establish horizontal control for related follow on topics. This will be done by follwoing the procedures set forth in this instructor guide and Engineering Aid 3 & 2 for traingulation and reducing traverses. All student responses will be 100% accurate for 3 order surveying precision.

Topic 1.2.3 Vertical Control Contact Hours: 7

Enabling objectives: Upon completion of this topic each student will while working as a member of a survey party and performing all duties of instrumentation, rodman and notekeeper; establish vertical control for the following topics of topographic and road surveying. All surveying procedures will be as specified in Engineering Aid 3 & 2, and this instructor guide. Student results will be to third order with 100% accuracy for procedures and calculations.

Topic 1.2.4 Topographic Survey Contact Hours: 10

Enabling Objectives: The student while working as a member of a survey party performing the duties of instrumentman, rodman, and notekeeper will develop a topographic map by use of statia, trigonometric leveling and the controlling point system. Procedures will be as set forth in this instructor's guide and Engineering Aid 3 & 2, NAVPERS 10634-B. All student responses will be 100% accurate for 3 order surveying precision.

Topic 1.2.5 Road Surveying Contact Hours: 9

Enabling Objectives: Upon completion of this topic each student will have performed the procedures involved in road surveying for location and layout. Each student will act as chairman, rodman, notekeeper and instrumentman in meeting the requirements of this topic. All procedures will be as outlined in this topic and Engineering Aid 3 & 2 NAVPERS 10634-B, Chapter 16. All survey work will be to third order accuracy, all profiles to the nearest 1/10 foot and all cross sections to the nearest 1/10 foot. All math computations will be 100% percent accurate as checked.

Topic 1.2.6 Road Surveys Contact Hours: 6

Enabling Objectives: Upon completion of this topic each student while working as a member of a survey party and performing the duties of instrumentman, rodman, notekeeper, and chainman will stake out the road designed in topic 1.2.5, "Road Surveying". Students will follow the procedures established by this instructor's guide and Engineering Aid 3 & 2 for setting centerline and related grade stakes and slope stakes. All student field work will be 100% for procedures and to third order for surveying accuracy.

Topic 1.2.7 Horizontal Curves Contact Hours: 12

Enabling Objectives: Upon completion of this topic the student will be able to design a horizontal curve and set up field notes for use by a survey party; consisting of four (4) students who will perform all the duties of an instrumentman, chainma, and notekeeper in laying out the curve on the road centerline established by topics 1.2.5 "Road Surveying", and 1.2.6 "Road Stakes". Design and layout procedures and computations will be as specified in this instructor's guide and Engineering Aid 3 & 2 NAVPERS 10634-B. Students will be 100% correct for all computations and procedures for designing the curve and in setting up the field notes; all field work performed by the survey party will be to 3rd order precision.

Topic 1.2.8 Vertical Curves Contact Hours: 7

Enabling Objectives: Upon completion of this topic each student will be able to a vertical curve and set up field notes for use by a survey party; consisting of four (4) students who will perform all the duties of an instrumentman, chainman and notekeeper; in laying out the vertical curve on the road centerline established by topics 1.2.5 "Road Surveying", and 1.2.6 "Road Stakes". Design and layout procedures and computations will be as specified in this instructor's guide and Engineering Aid 3 & 2, NAVPERS 10634-B. Students will be 100% correct for all computations and procedures for designing the vertical curve and in setting up the field notes; all field work performed by the survey party will be to 3rd order precision.

Topic 1.2.9 Grade and Earthwork Computations Contact Hours: 7

Enabling Objectives: Upon completion of this topic the student will have fixed grade as specified by the instructor, computed earthwork volumes using both the AVERAGE END AREA and PRISMOIDAL FORMULA methods and draw the mass diagram. The student will use the rough profiles and cross sections from topic 1.2.4 "Road Surveying", instructor guidance and Engineering Aid 3 & 2, NAVPERS 10634-B in meeting the requirements of this topic. Each student will perform his own calculations. All math computations will be 100% percent, all earthwork volumes as individually calculated, will be within 5% percent as a party for accuracy.

Topic 1.2.10 **Engineers** Transit Adjustment Contact Hours: 5

Enabling Objectives: Upon completion of this topic each student will be able to perform the proper sequence of transit adjustment and test in the field. The student reference materials and guidance of the instructor may be utilized in the performance of the requirements of the topics found in the SCBT 410.2 EA JS 1.2.10.1. Students will test each adjustable transit part until such errors are neutralized, and will be capable of attaining a third order accuracy when used in survey.

Topic 1.2.11 Level Adjustment Contact Hours: 5

Enabling Objectives: Upon completion of this topic each student will be able to adjust the dumpy, wye and self leveling levels. Procedure for testing and adjusting levels shall be set forth by this instructor's guide and Engineering Aids 3 & 2, NAVPERS 10634-B. Students shall test and adjust the levels until all errors are neutralized and the instrument is capable of attaining 3rd order accuracy when used in surveying operations.

Topic 1.2.12 Utility Surveying Contact Hours: 7

Enabling Objectives: Upon completion of this topic the student will be able, while acting as a member of a survey party, perform a utility survey, staking out a gravity flow sewer line. Procedures will be as outlined in this topic, Engineering Aid 3 & 2, NAVPERS, 10634-B. Student work will be accurate to 0.01 of a foot per 25 foot station.

Topic 1.2.13 Building Layout Contact Hours: 10

Enabling Objectives: Upon completion of this topic each student will be able to layout a married pre-engineered steeling building (2-40' x 100') while acting as a chainman, rodman, instrumentman and notekeeper in a survey party. Procedures will be as outlined in Engineering Aid 3 & 2, NAVPERS, 10634-B, Chapter 16, page 532 through 534 and Manufacturer's Drawings and Specifications. Student application will be checked by measuring the diagonal with an accuray of ± 1/16 inch as measured on the diagonal and checked against each other and the computed diagonal distance.

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ANNEX I

TEXTS

1. Engineering Aid 3 & 2, NAVPERS 10634-B
2. Engineering Aid 1 & C, NAVEDTRA 10635-B

ANNEX II

REFERENCES

1. NAVCONSTRACEN Instruction 5400.4
2. NAVMAT P-5100, "Safety Practices for Shore Activities".
3. Engineering Aid 1 & C, NAVEDTRA 10635-B.
4. Engineering Surveys (Elementary and Applied) Rubey, Lommel, Todd.
5. Surveying: Theory & Practice, 5th Edition. Davis, Foote and Kelly.
6. Surveying, Legault, McMaster, Marlette.
7. Surveying Practice, Philip Kissan.
8. NMCB Table of Allowance, Kit 800/0
9. Surveying Theory and Practices, Tracy.
10. Elementary Surveying, Breed & Hosmer.

ANNEX III

TOOLS, EQUIPMENT AND MATERIALS:

EQUIPMENT:

1. Kit Surveyor Equipment F/4-Men

COG STOCK NUMBER	ITEM	QUAN.	COST	EXT. COST
9QG 5120-203-4656	Hammer Engnrs 2 1/2 lb.	1 ea.	$ 1.80	$ 1.80
9QG 5120-900-6096	Hammer Sldg Dbl FC 8 lb. w/Fiberglass handle.	1 ea.	7.53	7.53
9QG 5210-224-8794	Plumb Bob Surv 16 oz.	4 ea.	2.83	11.32
9QG 5210-243-2878	Tape Measure Steel 300 ft.	1 ea.	40.00	40.00
9QG 5210-267-1949	Clamp Handle Stl Tape	2 ea.	2.83	5.66
9QG 5210-273-1958	Sheath Plumb Bob	4 ea.	1.18	4.72
9QG 5210-288-8759	Repair Sect 5/16 Stl.tape	1 Bx.	.76	.76
9QG 5210-293-0041	Repair Kit Stl Meas Tape	1 ea.	27.30	27.30
9QG 5210-293-2295	Tape Meas Stl Survey Open Face 100 Ft.	2 ea.	13.75	27.50
70Z 5210-469-8117	Rule Eng Wood Folding 6 Ft.	4 ea.	2.50	10.00
70Z 5210-488-0365	Tape Meas Fbrglas 100 Ft.	1	9.00	9.00
9ZD 5315-161-9815	Tack Stake	14 Hd.	.08	1.12
9ZD 5315-543-3021	Nail Drive Screw in F/Concrete Masonry 5 lb. pg.	1 Pg	2.50	2.50
9ZD 5315-543-3022	Nail Drive Screw 2 3/4 N	1 Pg.	1.90	1.90
1HB 6605-846-7618	Compass Lensatic Dry W/CS	2 ea.	10.50	21.00
9GD 6675-245-6680	Arrow Survey Rd-Wht 14 lg.	1 Se.	3.50	3.50
9GD 6675-281-2123	Surveyors Lthr	1 ea.	18.00	18.00
9GL 6675-470-2608	Target Plumb Bob Cord Pls.	6 ea.	.50	3.00
9GL 6675-[illegible]	Pole Range Survey 6 1/2 Ft.	4 ea.	5.38	21.52

TOOLS, EQUIPMENT AND MATERIALS (Cont'd)		QUAN	COST	COST
9GD 6675-641-3163	Level Survey Hand 6 1/2 Abney	2 ea	27.50	55.00
9GD 6675-641-5713	Rod Level G N-Flx 7F	2 ea	32.20	64.40
9GD 6675-802-9750	Transit Survey w/Tripod	1 ea	671.00	671.00
9GD 6675-869-8053	Level Survey w/Tripod	1 ea	507.00	507.00
9GG 7510-161-5675	Crayon Lumber Blue	2 dz.	.45	.90
9QG 7510-189-7881	Pencil Drawing 3H	1 dz.	.35	.35
9QL 7510-634-6516	Crayon Lumber Red	2 dz.	.49	.98
9QG 7530-263-2829	Book Engineer Trans	6 ea.	2.37	14.22
9QL 7530-263-2830	Book Engineer Level	6 ea.	1.03	6.18
9QG 9905-542-4504	Ribbon Flagging Surv Red	6 Ro.	.25	1.50
9QG 9905-684-4646	Ribbon Flagging Surv Wht	6 Ro.	.23	1.38
MISCELLANEOUS MATERIALS:				
	File	6 ea.		
	Bull Point	6 ea.		
	Machete	6 ea.		
	Thermometers	6 ea.		
	Graph Paper	3 ro.		
	Engineer's Scale	12 ea.		
	Architect's Scale	12 ea.		
	Profile paper	4 ro.		

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ANNEX IV

TRAINING AIDS:

Transparencies:

1. 12-11017.1T-22, Two peg test.
2. 12-11017.1T-31, Different types of crosshairs.

Locally Prepared Materials:

1. Data Sheets.
    a. SCBT 410.2 EA DS 1.2.10.1, Building Layout.
2. Information Sheets.
    a. SCBT 410.2 EA IS 1.2.3.1, Vertical Control.
    b. SCBT 410.2 EA IS 1.2.7.1, Horizontal Curves.
    c. SCBT 410.2 EA IS 1.2.7.2, Arc and Chord Definition.
    d. SCBT 410.2 EA IS 1.2.7.3, Horizontal Curve Formulas.
    e. SCBT 410.2 EA IS 1.2.7.4, Sample Note Format (Horizontal Curve)
    f. SCBT 410.2 EA IS 1.2.7.5, Curve Computation and Correction Tables.
    g. SCBT 410.2 EA IS 1.2.8.1, Sample Note Formats (Vertical Curve)
    h. SCBT 410.2 EA IS 1.2.9.1, Volume Determination
    i. SCBT 410.2 EA IS 1.2.9.2, Example Converted Soil Volumes.
    j. SCBT 410.2 EA IS 1.2.9.3, Mass Diagram
3. Job Sheets
    a. SCBT 410.2 EA JS 1.2.10.1, Transit Adjustment
4. Problem Sheets.
    a. SCBT 410.2 EA PS 1.2.2.1, Adjusting Angles
    b. SCBT 410.2 EA PS 1.2.2.2, Traverse Computations

Problem Sheets (Cont'd)

c. SCBT 410.2 EA PS 1.2.4.1, Contour Interpolation

d. SCBT 410.2 EA PS 1.2.8.1, Vertical Curves

ANNEX V

TRAINING AIDS EQUIPMENT

Projectors:

1. Overhead.

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ANNEX VI

FORMS:

1. None

ANNEX VII

MASTER SCHEDULE

FIRST WEEK

TOPIC NO.	TYPE	PERIOD	TITLE	RATIO
FIRST DAY				
1.1.1	C	1	Introduction and Safety	12/1
1.2.1	C	2	Construction Surveying	12/1
		3		
		4		
1.2.2	C	5	Horizontal Control	12/1
		6		
		7		
SECOND DAY				
1.2.2	P	8	Horizontal Control	12/1
		9		
		10		
		11		
1.2.3	C	12	Vertical Control	12/1
	P	13		
		14		
THIRD DAY				
1.2.3	P	15	Vertical Control	12/1
		16		
		17		
		18		
1.2.4	C	19	Topographic Survey	12/1
		20		
		21		
FOURTH DAY				
1.2.4	P	22	Topographic Survey	12/1
		23		
		24		
		25		
		26		
		27		
		28		
FIFTH DAY				
1.2.5	C	29	Road Surveying	12/1
		30		
	P	31		12/1
		32		

SECOND WEEK

TOPIC NO.	TYPE	PERIOD	TITLE	RATIO
FIRST DAY				
1.2.5	P	1	Road Surveying	12/1
		2		
		3		
		4		
		5		
1.2.6	C	6	Road Surveys	12/1
		7		
SECOND DAY				
1.2.6	P	8	Road Surveys	12/1
		9		
		10		
		11		
1.2.7	C	12	Horizontal Curves	12/1
		13		
		14		
THIRD DAY				
1.2.7	P	15	Horizontal Curves	12/1
		16		
		17		
		18		
		19		
		20		
		21		
FOURTH DAY				
1.2.7	P	22	Horizontal Curves	12/1
		23		
1.2.8	C	24	Vertical Curves	12/1
		25		
	P	26		12/1
		27		
		28		
FIFTH DAY				
1.2.8	P	29	Vertical Curves	12/1
		30		
1.2.9	C	31	Grade and Earthwork Computations	12/1
		32		

THIRD WEEK

TOPIC NO.	TYPE	PERIOD	TITLE	RATIO
FIRST DAY				
1.2.9	P	1	Grade and Earthwork Computations	12/1
		2		
		3		
		4		
		5		
1.2.10	C	6	Engineering Transit Adjustment	12/1
		7		
SECOND DAY				
1.2.10	P	8	Level Adjustment	12/1
		9		
		10		
1.2.11	C	11	Transit Adjustment	12/1
		12		
	P	13		12/1
		14		
THIRD DAY				
1.2.11	P	15	Level Adjustment	12/1
1.2.12	C	16	Utility Surveying	12/1
		17		
	P	18		
		19		
		20		
		21		
FOURTH DAY				
1.2.12	P	22	Utility Surveying	12/1
1.2.13	C	23	Building Layout	12/1
		24		
	P	25		
		26		
		27		
		28		
FIFTH DAY				
1.2.13	P	29	Building Layout	12/1
		30		
		31		
		32		

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
ENGINEERING AID SCBT 400-490

Classification: Unclassified

Topic: Introduction and Safety

Average Time: 1 Period (Class)

Instructional Materials:

A. Texts: None

B. References:

1. NAVCONSTRACEN Instruction 5400.4 (Current Series), Organization Manual of NAVCONSTRACEN
2. "Safety Practices for Shore Activities", NAVMAT P-5100, (Jan 73)

C. Tools, Equipment and Materials:

1. See Annex III of Curriculum.

D. Training Aids and Devices:

1. None.

E. Training Aids Equipment:

1. None.

Terminal Objective: Upon completion of this unit the student will have registered for the course, received text books, complied with NAVCONSTRACEN and CBC regulations governing the reporting and fighting of fires which pertain to him as a SCBT student.

Enabling Objective: Upon completion of this topic the student will be able to answer orally specific questions pertaining to the mission, regulations and organization of the Command, and the method of reporting and fighting fires as established by NAVCONSTRACEN and CBC regulations.

Criterion Tests: The student will answer orally specific questions pertaining to the mission, regulations and organization of the Command, and the method of reporting and fighting fires as established by NAVCONSTRACEN and CBC Regulations

Homework: None

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. INTRODUCTION TO THE LESSON	1. Introduce self and topic.	
A. Establish Contact.		
1. Name:		
2. Topic: Introduction and Safety		
B. Establish Readiness	Motivate student.	
1. Purpose		
2. Assignment		
C. Establish Effect	3. Bring out need and value of material being presented.	
1. Value		
a. Pass course.		
b. Perform better on the job.		
D. Overview:	4. State learning objectives:	
1. You will be able to answer orally specific questions related to the mission, regulations and organization of the Command, and the methods of reporting and fighting a fire as established by NAVCONSTRACEN and CBC regulations.	a. State information and materials necessary to guide student.	
2. Ask questions.		
3. Take notes		
4. Testable		

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

II. PRESENTATION:

A. Introduction

1. Mission

a. Special training course

b. Higher state of readiness,

c. Compliance with COMCBPAC Instruction.

2. Organization and Chain of Command

a. Commanding Officer

b. Executive Officer

c. Training Officer

d. School Department Officer

e. Division Director

f. Senior Instructor

h. Class Petty Officer

3. Regulations and policies

a. Break procedures

b. Uniform regulations

(1) Working uniform of the day.

(a) Must be neat and clean.

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

c. Absenteeism

(1) Must be kept to a minimum.

(2) Medical or dental sick call.

(3) Permission to be absent.

d. Parking

(1) Where

(2) When

(3) How

e. Visitors and phone calls

(1) Emergencies only.

(2) Phone numbers

(a) School number

f. Lost or damaged material

(1) Text books

(2) Publications

(3) Tools

(4) Materials

(5) Statement of charges

g. Clean-up procedures

OUTLINE OF INSTRUCTION INSTRUCTOR ACTIVITY STUDENT ACTIVITY

h. Problems
    (1) Scholastic
    (2) Personal
    (3) Counseling assistance

4. Standards of student performance
    a. Written examinations
    b. Homework assignments
    c. Practical application

5. Course outline
    a. Course mission
    b. Course objectives
    c. Reading assignments
    d. Class schedule

6. Grading system
    a. Homework
    b. Practical application
    c. Quizzes
    d. Weekly tests
    e. Final examination

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
B. Safety		
1. Reporting accidents		
a. Class safety man	B.1.a. Pick safety man and explain job.	
b. Instructor		
c. School director		
d. First aid when appropriate.		
2. Fire safety		
a. Evacuation routes		
b. Reporting fires		
c. Fighting fire		
(1) Location of extinguishers.		
III. APPLICATION:		
A. Discussion	III.A. Questions To be developed by the instructor.	III.A. Answers
IV. SUMMARY:		
A. Introduction		
1. Mission		
2. Organization and Chain of Command		
3. Regulations and Policies		

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

4. Standards

5. Course Outline

6. Grading System

B. Safety

1. Reporting accidents

2. Fire safety

V. TEST:

A. None

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Construction Surveying

Average Time: 3 Periods (Class) 0 Periods (Pract)

Instructional Materials:

A. **Texts:**

1. **Engineering** Aid 3 & 2, NAVPERS 10634-B.

2. **Engineering** Aid 1 & C, NAVPERS 10635-A.

B. References:

1. **Surveying** - Legualt, McMaster, Marlette.

2. Engineering Surveys (Elementary and Applied) Rubey, Lommel, Todd.

3. Surveying Practice, Philip Kissam.

4. NMCB Table of Allowance, kit 800/0.

C. Tools, Equipment and Materials:

1. NMCB TOA 00405A/80010 Kit, Surveyors Equipment.

D. Training Aids and Devices:

1. None.

E. Training Aids Equipment:

1. None

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will identify the types of construction surveys, basic surveying procedures and safety practices. Be issued surveying equipment on a party basis. In order to perform these requirements the student will use the student texts, Naval Mobile Construction Battalion Table of Allowance Kit 0045A/80010, Surveyors Equipment and instructor guidance. The minimum student requirements will be evaluated on his performance in the field for basic surveying procedures and safety practices while performing construction surveys required by the remaining topics of this unit and 100% percent accuracy when inventorying party kits.

Criterion Test: Each student will be evaluated upon his performance of basic surveying procedures and safety practices when performing the requirements of the remaining topics of this unit and 100 percent accuracy when inventorying party kits.

Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B. pp.

Engineering Aid 1 & C, NAVPERS 10635-A, pp.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Construction Surveying.		
B. Establish readiness.	I.B. Motivate student	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to identify the types of construction surveys, basic surveying procedures and safety practices and be issued kit,"surveyor's equipment"to be used for practical application.	I.D.1. State information and materials necessary to guide student.	
2. Take notes.		
3. Ask questions		
4. Testable.		

OUTLINE OF INSTRUCTION	**INSTRUCTOR ACTIVITY**	**STUDENT ACTIVITY**

II. Presentation.

A. Construction Surveying Categories.

1. Area Surveying.

    a. Construction projects in an area.

        (1) Buildings, dams, bridges, airfields, etc.

    b. Involves two basic problems.

        (1) Obtaining data from the field to develop a plot plan.

        (2) Laying out of a project in the field from an existing plot plan.

2. Route surveying.

    a. Construction projects in length.

        (1) Roads, canals, pipe lines, power lines, etc.

    b. Must have starting and finishing points.

        (1) May or may not be exactly defined.

            (a) During reconnaissance phase points may be only roughly defined.

(b) Upon completion of survey points will be defined using an exact method, commonly by latitude and departure.

c. Route surveys normally involve four different subdivisions.

(1) Reconnaissance - made to determine general conditions.

(a) Is a rapid and rough survey or examination of the territory involved in a proposed project.

(b) Steps.

1. Secure best maps.

2. Go over area.

3. Determine elevations.

4. Study support features.

(2) Preliminary survey - a more careful instrumental survey or surveys of selected routes.

(a) Made by transit traversing or compass traversing or serial surveys.

(b) Run so that it provides a base for narrow topographic maps.

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

(c) Older preliminary surveys included travers party, level party and topographic party.

(d) Precision is as required for topographic maps.

(3) The final location survey gives the exact location of the center line.

(a) Usually run with a precision of from 1/2000 to 1/5000.

(b) Minor location changes as required by field conditions are made at this time.

(c) Curves layout.

(d) Profiles and cross sections are run.

(e) Detailed plans are worked up now.

(4) Construction surveys - controls actual construction.

(a) Sets additional references.

(b) Sets all construction stakes.

(c) Inspects and passes on the construction work.

d. In this class we will deal primarily with steps 3 and 4.

(1) Class projects will include.

(a) 2 (40' x 100') R.F. pre-engineered building with a T shape.

(b) You will layout a road which will include a horizontal and vertical curve, slope stakes, ditches and other related work and computation.

B. Basic surveying review.

1. Accuracy of surveys.

a. Precision varies with type of work involved.

b. Usually determined by Officer or Petty Officer-in-Charge.

c. Type of measurement requires the surveyor to make practical analysis and choose the appropriate methods and procedures.

(1) Allowable time.

(2) Situation.

(3) Capabilities of construction workers.

(4) Prevailing conditions.

d. The best surveyor runs the job to the order of precision required with minimum of wasted time not the one who requires extreme precision at all times.

2. Field notes.

a. Quality and character of field notes are as important as a surveyors use of instruments.

b. Competence measured by the comprehensiveness, neatness, and reliability.

c. Notes, sketches and numerical data can only be interpreted in one way, the correct way.

d. Office entries often made in red, must be different.

e. Good rules to follow when taking field notes are:

(1) Use a well pointed pencil.

(2) Do not crowd data.

(3) Keep sketches plain and uncluttered.

(4) Record numerical values to required, example: nearest 0.01 not 5.3 feet but 5.30 feet.

(5) Use explanatory notes to supplement numerical data and sketches.

(6) Follow correct format.

3. The important considerations in field work.

a. Study of the field problem.

(1) Mentally go through the various steps involved by asking yourself these questions.

(a) What is the purpose of the survey.

(b) What degree of precision is required for that purpose.

(c) What type and precision of control must be established.

1. Is there any existing control that can be used.

(d) With what precision must each measurement be taken.

(e) What are the sources of errors.

(f) What methods must be employed to keep these errors within allowable limits.

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

(g) What instruments should be used to facilitate the work.

(h) How may the work be organized to reduce the labor to a minimum.

(i) How is the correctness of the work to be verified.

b. Prepare a list of necessary equipment.

(1) Important.

(a) Facilities getting equipment together.

(b) Avoid the dilemna of being caught miles away from the office without a necessary piece of equipment.

(2) Examine equipment before going into the field.

(a) Student have taken broken equipment out with them.

(b) And other goofs such as taking out a tripod with threading that did not match that of the instruments.

c. Speed.

(1) Secondary to accuracy as far as students are concerned.

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(a) However, most students are just too slow due to lack of self confidence and/or lack of attention to the task.

1. Can be remedied by the following (3b and 3c).

(2) Requires practice in handling the instruments.

(a) So at every opportunity practice the proper mechanics in handling all of the instruments.

1. Come in for night study to get that extra practice most of you need because the time in the field during the day is limited.

2. Students have a tendency to work with only certain instruments for personal reasons and fail to develop the necessary skills in the use of all instruments.

(b) Secrets of successful use of an instrument.

1. Formation of a set of standard habits in the way you use the instruments.

2. Clear knowledge of exactly what the instrument is and what it can do.

3. An understanding of the ever-present necessity for speed.

(3) Work must be carefully planned and systematized.

(a) Most important factor in achieving speed and satisfactory results.

1. Know what your are going to do.

2. How you are going to do it.

3. Then do it.

d. Habit of correctness.

(1) No measurement correct until verified.

(a) This is one attitude and habit that no engineer can do without.

(b) Mistakes in engineering involve too much time, labor and material (in one work-money) to correct.

- (2) Method of check should differ from original method of measurement.
    - (a) Not likely to make the same mistake over and over.
- (3) All persons are liable to make mistakes.
    - (a) We all have our bad days.
    - (b) However, don't allow any other person than yourself discover the discrepancy.
- (4) Habitual carelessness injurious to your reputation.
    - (a) Even after one mistake, people begin to doubt that any of your work is or ever will be correct.

e. Consistent precision.

- (1) Precision of measurements should be consistent with the purpose of the survey.
    - (a) Learn to comprehend the different types of work.
    - (b) Maintain a consistent degree of precision throughout each survey.

(2) There is no fixed rule for the relative precision of the different classes of surveys.

(a) The objects and conditions are too many and too complicated.

(b) So if precision is not given for your particular survey, use common sense and mathematics.

f. Requisites of a good surveyor.

(1) A thorough knowledge of the theory of surveying and skill in the practice are the principle requisites.

(2) Yet it is also true that traits of character and habit of mind are far more potent factors in the success of the engineer than is more technical knowledge of skill.

(3) Some of the traits of character and habits of mind which are essential to an engineer.

(a) Maintain the attitude of the scientist, that no result is trustworthy until every reasonable test of its accuracy has been applied.

(b) Be reliable.

(c) Possess initiative and attack each problem with resourcefulness and energy.

(d) Be thorough, not content with your work until it has been finished in a workman like fashion.

(e) Be of sound judgment, able to think without confusion and to reason logically without prejudice.

(f) Able to get along with people, thoughtful of those coming under his direction, commanding the respect of his associates, and watchful of the interest of his employer.

4. Sources of surveying information.

a. Military.

(1) Naval Facilities Engineering Command (Yards & Docks).

(a) JAN & MIL STDS.

(b) A & E drawings.

(c) Design manuals.

(2) Records of previous battalions.

- (a) Maps.
- (b) Survey data.
- (c) Reports.

(3) Public Works Department.

- (a) Maps.
- (b) Blue prints.
- (c) Survey data.

(4) Resident Officer-in-Charge of Construction (ROICC).

- (a) Jurisdiction.
- (b) Survey data.
- (c) Plans and specifications.

5. Gevernment agencies.

(1) Department of the Army.

- (a) Technical Manuals (TM).
- (b) Topographic maps.

(2) U.S. Coast and Geodetic survey.

- (a) Control data.
- (b) Maps and charts.

(c) Special publications.

(3) U.S. Navy Hydrographic Office.

(a) Charts.

(b) Hydro publications.

(4) National Bureau of Standards.

(a) Standardization of tapes.

c. Bureau of Land Management.

(1) Information concerning survey of public land of the United States.

(a) Regulations.

(b) Methods.

(c) Maps of previous surveys.

(d) Data of previous surveys.

d. Superintendent of Documents, U.S. Government Printing Office.

(1) Publishes and sells all the maps, data and instruction manuals of the above named organizations.

(2) Many of the maps, data and instruction manuals are at no cost when ordered through the Supply Department.

5. Safety in surveying.

   a. The concern of every man in the party.

      (1) Don't feel it's only the supervisors and safety man's responsibility.

      (2) So notify everyone in the area of a hazard.

   b. Some of the danger points.

      (1) Sharp tools.

         (a) Safer when kept sharp.

         (b) Sheaths are desirable for carrying from one place to another.

         (c) When laid down temporarily, place in such a way so that injury cannot result to anyone.

         (d) When using cutting tools, have regard for yourself and those around you.

   c. Reapir of tools.

      (1) Don't work with tools in need of repair.

         (a) Loose heads on any hand tools.

         (b) Unsharp cutting tools.

(c) Mushroomed heads on bull points.

d. Pointed tools.

(1) Examples.

(a) Range poles.

(b) Chaining pins.

(c) Machete.

(2) Don't play with them.

(a) Danger involved.

(b) Damaged equipment.

(3) Carry in such a manner as not to danger yourself or others.

(a) Know where the point is at all times.

e. Stowing equipment improperly.

(1) Don't lay equipment down anywhere it may cause an accident.

(a) Equipemnt laid where someone is likely to trip over them.

(b) Rods and range poles leaned against trees and buildings in such a manner they fall or are blown over.

(2) Stow equipment properly when not to be used for awhile.

f. Handling equipment improperly.

(1) Carry range poles and rods vertically against your body when near other people.

(2) Don't turn suddenly with equipment horizontally on your shoulder or in your hands.

g. Traffic.

(1) Let equipment operators know you are working in the area.

(2) Wear bright clothing or set out caution signs when working near a highway or runway.

(3) Have road guards or caution signs when surveying across a road or runway.

(4) Improper clothing.

(a) Wear a cap to protect yourself from the direct rays of the sun.

(b) Wear sunglasses as protection against glare.

(c) Wear the proper type of foot wear and stockings.

(d) Wear the type of clothing necessary to protect you from the weather and field hazards (snakes, vegetation, insects, etc.)

h. Knowledge of first aid essential.

(1) Reason - likely to be working at a considerable distance from a sickbay or hospital.

(2) First aid - you should know how to give.

(a) Sunstroke.

(b) Control bleeding.

(c) Snake bites.

(d) Set broken bones.

(e) Artificial respiration.

(f) Frostbite.

(g) Poisonous plants.

i. Important to carry a first aid kit in the field.

(1) Far from a sickbay or hospital.

(2) Safetyman responsibility while here at EA school.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACITVITY
(3) Party chief should have one made up for his crew to take with them in the field.		
j. Knowledge of nearest first aid station important.		
(1) Find out nearest doctor, hsopital (military or civilian) and sick-bay before taking crew into the field.		
(2) Know the telephone numbers of facilities where help is available.		
C. Surveying equipment.		
1. Establish parties and party chiefs.	II.C.1. Explain that each person will perform all functions related to the survey party.	
2. Issue survey gear.		
a. Issue kits.	II.C.2.a. Hold inventory, stress care of instruments and equipment. Pass out TOA print out.	II.C.2.a. Students inventory gear and check instruments for accuracy as per procedures taught in SCBT 410.1 Basic Surveying.
D. Review.		
1. Ask students if they have any questions pertaining to Basic Surveying before proceeding to application.		
III. Application.	III.A. Questions	III.A. Answers
A. Questions and discussion	1. What are the two types of construction surveys?	1. Area and route.

OUTLINE OF INSTRUCTION

IV. Summary

- A. Construction surveying.
  - 1. Area surveying.
  - 2. Route surveying.
- B. Basic surveying review.
  - 1. Accuracy of surveys.
  - 2. Field notes.
  - 3. The important considerations in field work.
  - 4. Sources of surveying information.
  - 5. Safety in surveying.
- C. Surveying equipment.
  - 1. Establish parties.
  - 2. Issue kits.
- D. Review.

V. Test.

- A. See Criterion Test.

INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
2. How is route surveying sub-divided?	2. Reconnaissance, preliminary, location and construction.
3. The best surveyors use what order of percision?	3. Whatever the job requires and no more.
4. Surveying notes should have what?	4. Quality & character.
5. What is considered important field work?	5. To study the problem, care and selection of equipment, speed, habit of correctness, consistent precision & a thorough knowledge of surveying.
6. What information would you find in the Public Works Department?	6. Maps, survey data, blue prints.
7. Who in a survey party is charged with safety?	7. Every man in the party.

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Horizontal Control

Average Time: 3 Periods (Class) 4 Periods (Pract)

Instructional Materials:

A. Texts.

1. Engineering Aid 3 & 2, NAVPERS 10634-B.

B. References:

1. Engineering Aid, 1 & C, NAVEDTRA 10635-B

2. Surveying - Theory and Practice, 5th Edition, Davis, Foote and Kelly.

C. Tools, Equipment and Materials.

1. See Annex III of Curriculum

D. Training Aids and Devices:

1. Films: None

2. Transparencies: None

3. Charts: None

4. Locally prepared Materials.

a. Practice Sheets.

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will; while working as a member of a survey party and performing all duties of instrumentman, rodman, notekeeper and chainman; establish horizontal control for related follow on topics. This will be done by following the procedures set forth in this instructor guide and Engineering Aid 3 & 2 for triangulation and reducing traverses. All student responses will be 100% accurate for 3 order surveying precision.

Criterion Test: Each student while working as a member of a survey party and performing all duties of instrumentman, rodman, notekeeper and chainman will establish horizontal control and a predetermined traverses net. All procedures will be as prescribed by instructional materials and Engineering Aid 3 & 2 for triangulation and reducing traverses. All student responses will be 100% accurate for 3 order surveying precision.

Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B, Chapter 13 all.

(1) SCBT 410.2 EA IG 1.2.2. Adjusting Angles.

(2) SCBT 410.2 EA IG 1.2.2. Traverse Computations

E, Training Aids Equipment.

1. None

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Horizontal Control.		
B. Establish readiness.	I.B. Motivate student	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value		
a. Pass course.		
b. Get advanced.		
c. Perform better on the job.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to establish horizontal Control.	I.D.1. State information and material necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

II. Presentation.

A. Horizontal Control.

1. The location in plan of control stations by means of triangulation and/or traversing.

   a. An extensive survey requires primary and secondary traverse system and/or triangulation system.

   b. Less extensive surveys requires only the primary system is necessary.

2. Traversing.

   a. Primary.

      (1) A relative term referring to the degree of precision used on a survey.

   b. Secondary.

      (1) Instrument stations from which details are located.

      (2) It may run simultaneously with/or separately from the survey for the location of details.

      (3) Usually run with a transit.

         (a) Plane table - location detail.

1. Avoid a large accumulation of errors.

3. Triangulation.

   a. Primary.

      (1) Field stations are established. on summits where visibility is good, and signals are erected.

      (2) One or more base lines are established and measured.

         (a) True azimuths are determined.

      (3) When the field measurements have been completed, the necessary computations and adjustments are made.

      (4) Coordinates of each station are computed for use in plotting.

   b. Secondary.

      (1) Employed in open country where chaining would be difficult.

      (2) Advantage - instrument stations can be chosen at strategic points.

      (3) Unaffected by the cumulative errors inherent in traversing.

4. Computing and distributing error of closure for traverse.

   a. Checking and reducing angles.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
(1) Begin traverse computations by checking to ensure that all the required angles (including closing angles) were turned, and that the notes correctly indicate their sizes.		
(2) For deflection angles, check to ensure that angles marked L or R were actually turned, and should have been turned, in those directions.		
(3) Check your sketches and be sure they are in agreement with your field notes.		
(4) Reduce repeated angles to mean angles.		
b. Adjusting angles.	II.A.4.b. Pass out SCBT 410.2 EA PS 1.2.2.4 and have students work.	II.A.4.b. Work problem sheet.
(1) In a closed traverse, the theoretical or geometrical sum of the interior angles is 180° x (n-2).		
(2) The sum of the exterior angles is 180° x (n+2).		
(3) The difference between the sum of the right deflection angles and the sum of the left deflection angles is 360°.		

60

(4) Any discrepancy between one of these sums and the actual sum of the angles as turned or measured constitutes the angular error of closure.

(5) You adjust the angles in a closed traverse by distributing an angular error of closure which is within the allowable maximum equally among the angles.

(6) Procedures for adjusting angles are outlined and examples are given in EA 1 & C, Chapter 5.

c. Compute bearings.

(1) Bearings.

(a) Definition.

1. System of designated direction.

(b) Measurement.

1. Horizontal angle, never more than 90°.

2. Measured from North and South.

3. Forward bearing.

a. Direction of survey

4. Back bearing

   a. 180° from forward bearing.

   b. With reverse quadrants.

   c. Will show any local attraction.

5. Correct for local attraction and convert to true bearing.

   a. Determine amount of local attration.

(2) Azimuths.

(a) Definition.

1. Angle measured clockwise from zero or 180°.

2. Greater than zero, but less than 360°.

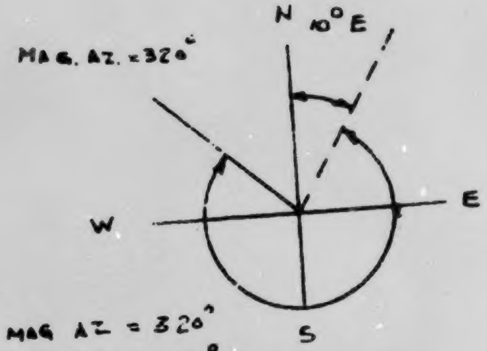
3. Plane surveying usually from North.

4. USC and GS from South.

5. True, magnetic or assumed.

   a. May be computed.

   b. Read on a transit circle.



MAG AZ = 320°
EAST DEC + 10°
TRUE AZ = 330°

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

6. Correct for local attraction and convert to true azimuth.

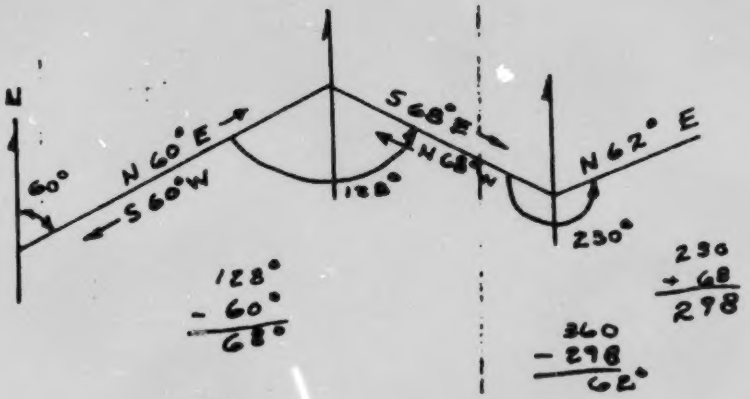
   a. East +

   b. West -

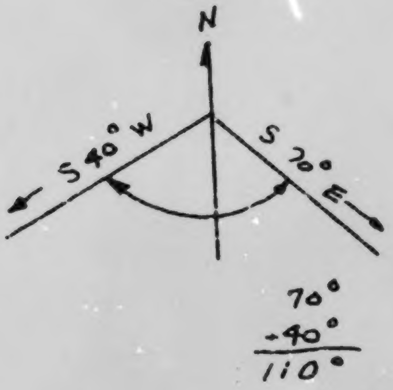
(3) Computations.

(a) Bearings and Azimuths.

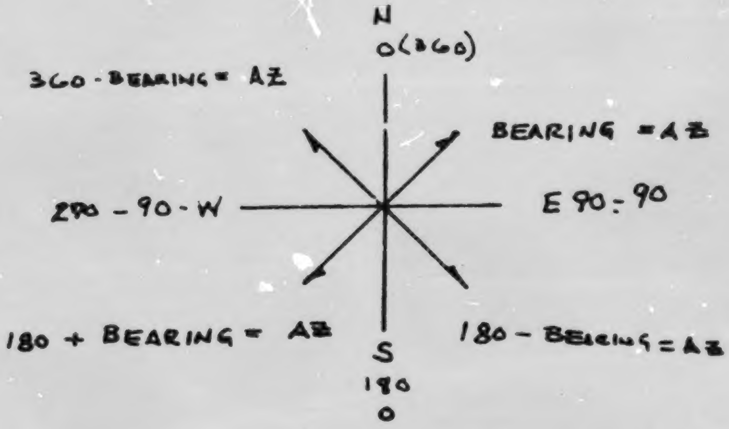
1. Given angles compute bearings.



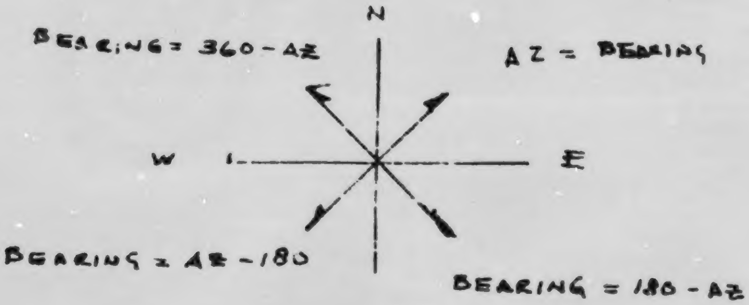
2. Given bearing, compute angle.



3. Given bearing, compute azimuth.

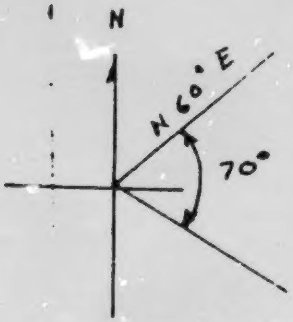


4. Given azimuth, compute bearing.



OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

5. Given one angle and one bearing, compute bearing and azimuth.



$\Delta Z = 60°$

$+ \ 70°$

$\Delta Z = 130°$

180

$- \ 130$

S 50° E

d. Checking and reducing distances.

(1) Check to ensure that all required linear distances have been chained.

(2) Reduce slope distances as required.

(3) If you broke chain on the slopes, check to ensure that sums of break distances were correctly added.

(4) Apply standard error, tension and temperature corrections as required.

e. Adjusting for linear error of closure.

(1) The procedure for distributing a linear error of closure (one within the allowable maximum, of course) over the directions and distances in a closed traverse is called "Balancing" or "Closing" the traverse.

(2) Before you can understand the procedures, you must have a knowledge of "Latitude" and "Departure".

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

f. Compute the latitude and departures.

(1) Latitude = Y = length multiplied by cosine of bearing.

N = plus

S - minus

(2) Departure = X = length multiplied by sine of bearing.

E = plus

W = minus

g. Compute the error.

(1) Begins and ends at same point, the sum of the latitude and sum of the departures should both equal zero.

$$TE = \sqrt{(Lats)^2 + (Departures)^2}$$

h. Compute the measure of accuracy or simply the accuracy.

(1) This is a ratio or the total error to the total length of the survey.

i. Adjust the latitude and departures.

(1) Reasonable process that makes them equal zero.

(2) Two methods.

(a) Compass rule - apply corrections in relation to the length of the course.

Cor. = $\frac{C}{L}$ S

Cor. = Correction

C = Total error with sign changed.

L = Total length of survey.

S = length of course.

(b) Transit rule - Apply correction in proportion to the length of the latitudes and to the departures in proportion to the length of the departures

Cor. = $\frac{C}{L}$ S

C = Total error in sum with sign changed.

L = Sum or latitudes (or departures) with out regard to sign.

S = Length of particular latitude (or departure).

(3) Correction must equal zero.

j. Compute the latitudes and departures. Add the corrections algebraically to the adjusted latitudes or departures.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
k. Compute the coordinates if required.		
l. Final operation.		
(1) Plotting the traverse.		
(a) Protractor and scale.		
(b) Coordinates - if known this is most accurate method.		
(c) Covered in greater detail in topographic surveying.		
m. Precision of control traverses.		
(1) Discuss Engineering Aid 3 & 2, NAVPERS 10634-B, Chapter 13, Page 447.		
III. Application		
A. Discussion	III.A. Questions	III.A. Answers
	1. In a primary triangulation net stations are usually set where?	1. On summits where visibility is good and then signals are erected.
	2. A secondary triangulation net is usually used where?	2. In open country where chaining would be difficult.
B. Practical performance.	III.B. Each student will act as instrumentmen, note keeper and rodman in a survey party that is establishing horizontal control over a predetermined course:	

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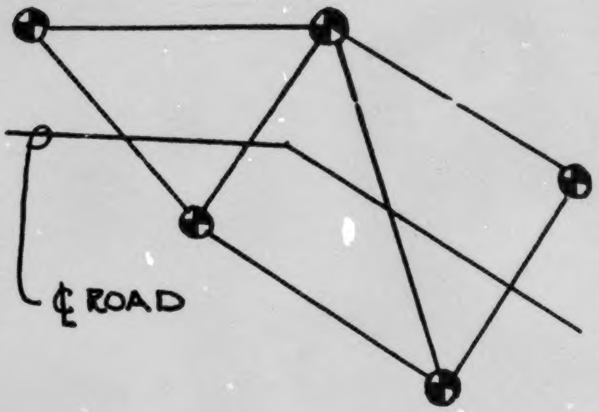
OUTLINE OF INSTRUCTION

INSTRUCTOR ACTIVITY

STUDENT ACTIVITY

Suggested Course:



This problem should be worked in conjunction with vertical control, road surveying, and topographic surveying problems.

IV. Summary

A. Horizontal Control

1. Definition

2. Traversing

V. Test: None

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

PROBLEM SHEET

TITLE: Adjusting angles.

OBJECTIVES: The purpose of this problem sheet is to develop your skills in adjusting angles for traverse computations.

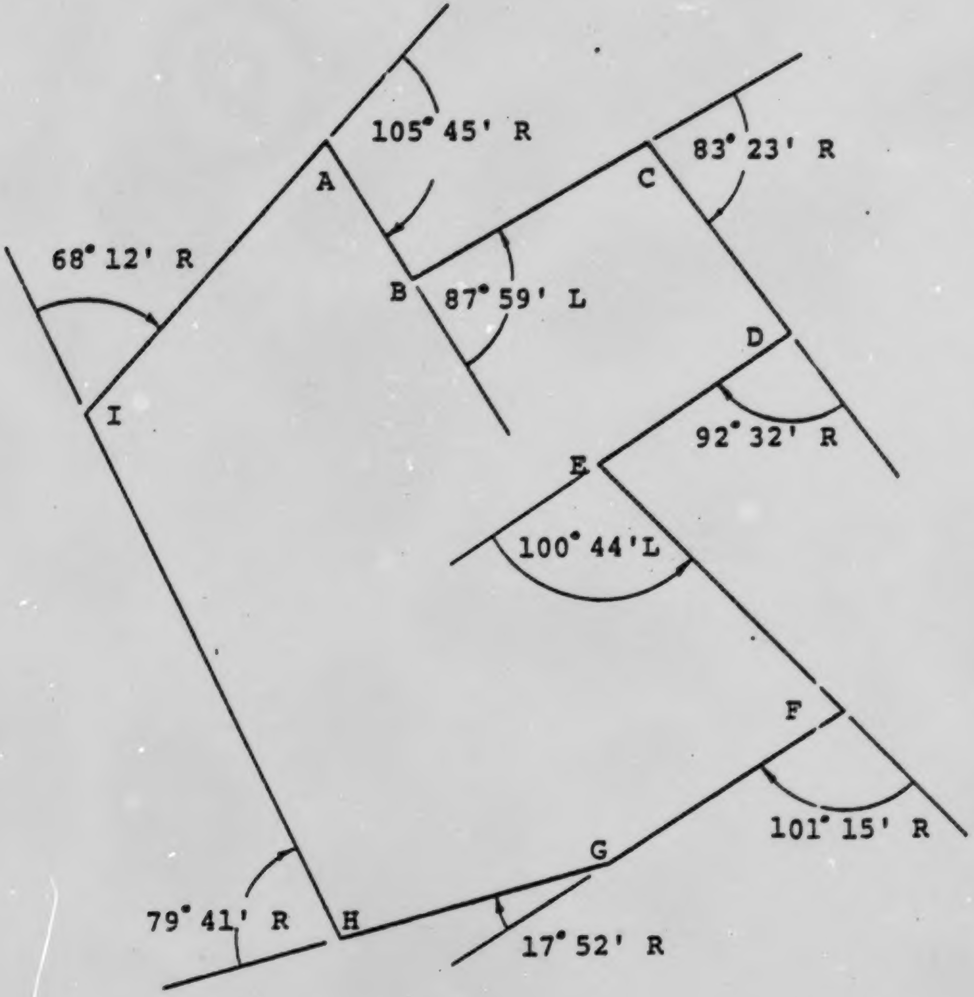
PROCEDURES:

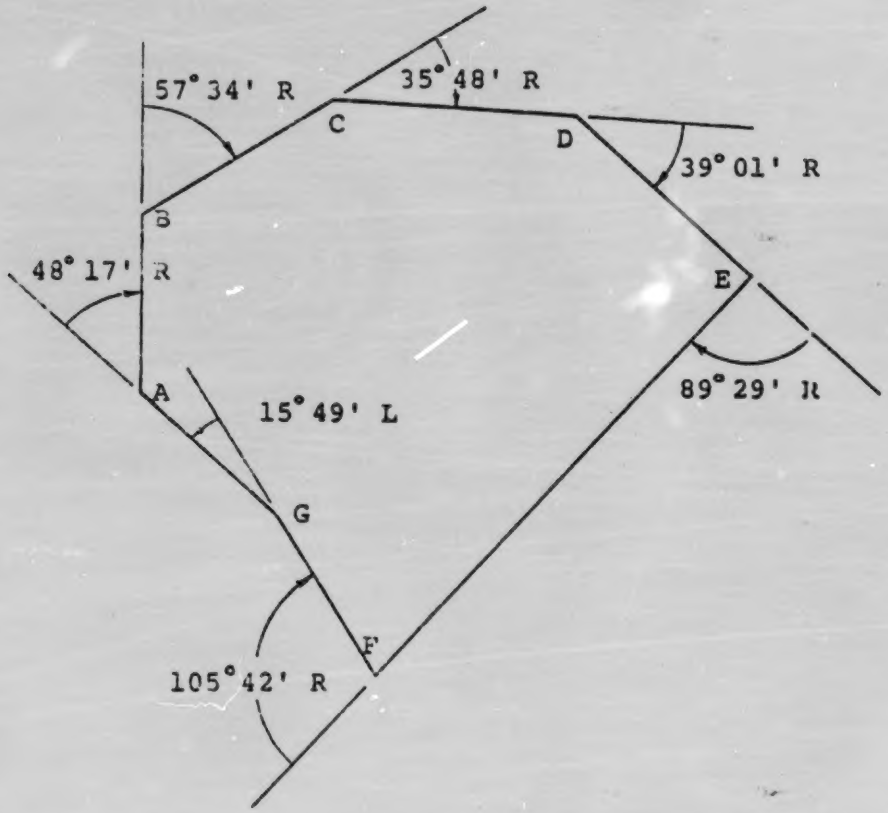
1. This problem sheet consists of five traverses which contain unadjusted angles. Adjust the angles in each closed traverse and distribute the angular error as outlined by the instructor. If you experience difficulties raise your hand and the instructor will assist you. Turn in the problem sheet to the instructor by ______________________.

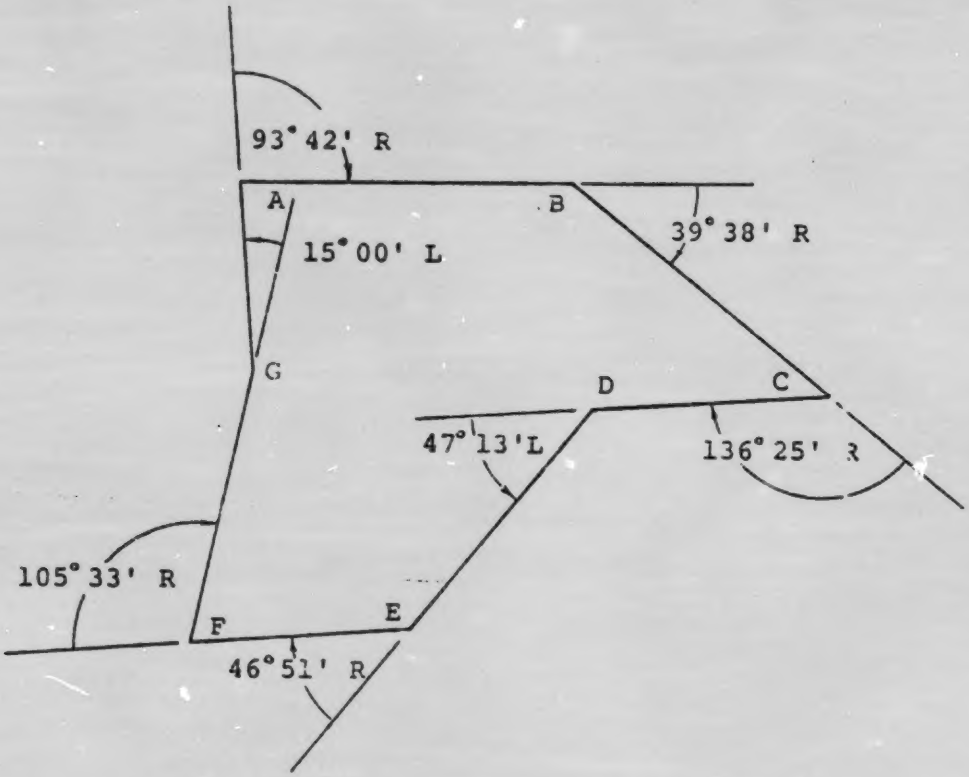
NAME: ______________________

BATTALION/CLASS NO: ______________________

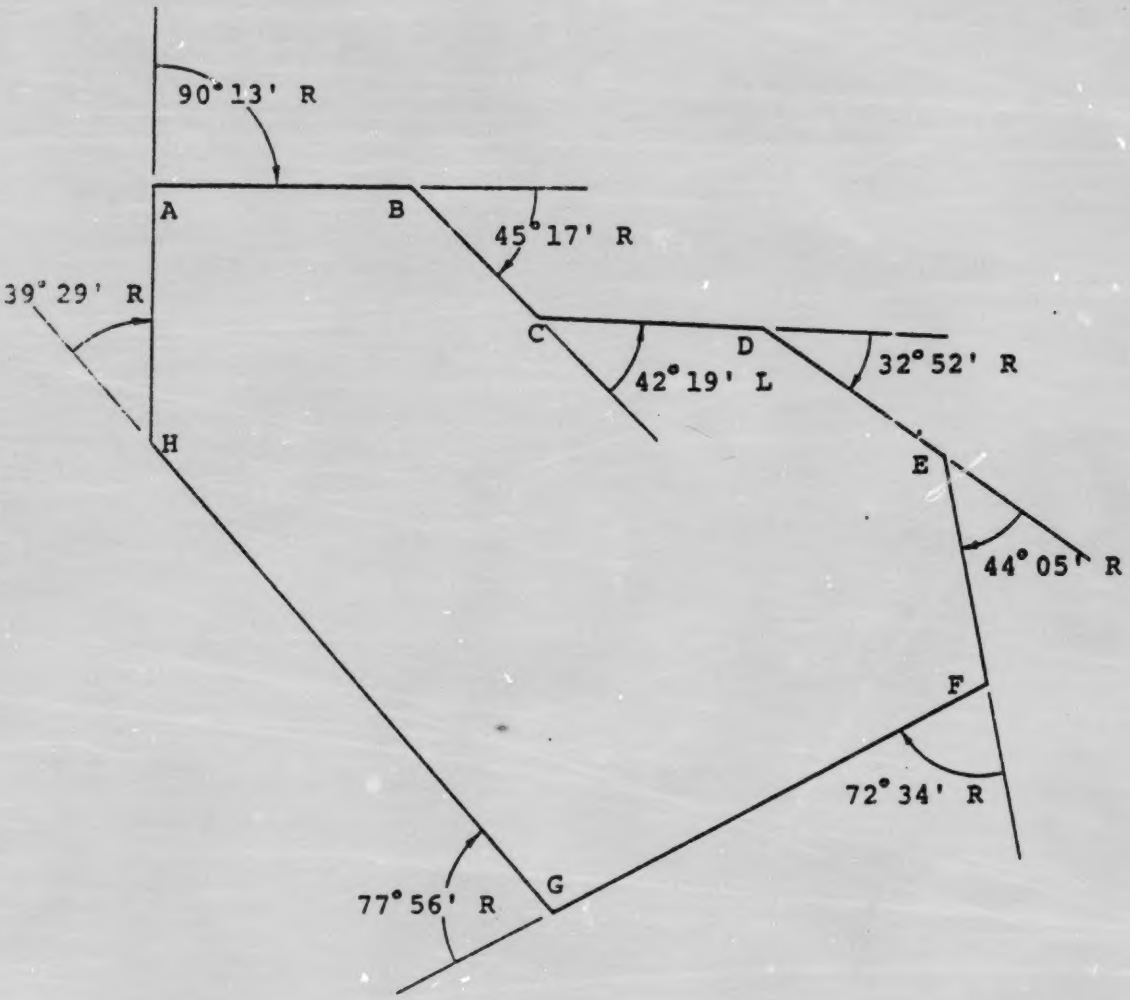
DATE: ______________________


105° 45' R
83° 23' R
A
C
68° 12' R
B
87° 59' L
D
I
92° 32' R
E
100° 44'L
F
101° 15' R
G
79° 41' R
H
17° 52' R


57°34' R
35°48' R
C
D
39°01' R
B
48°17' R
E
A
15°49' L
89°29' R
G
F
105°42' R


93° 42' R
A
B
39° 38' R
15° 00' L
G
D
C
47° 13'L
136° 25' R
105° 33' R
F
E
46° 51' R

73



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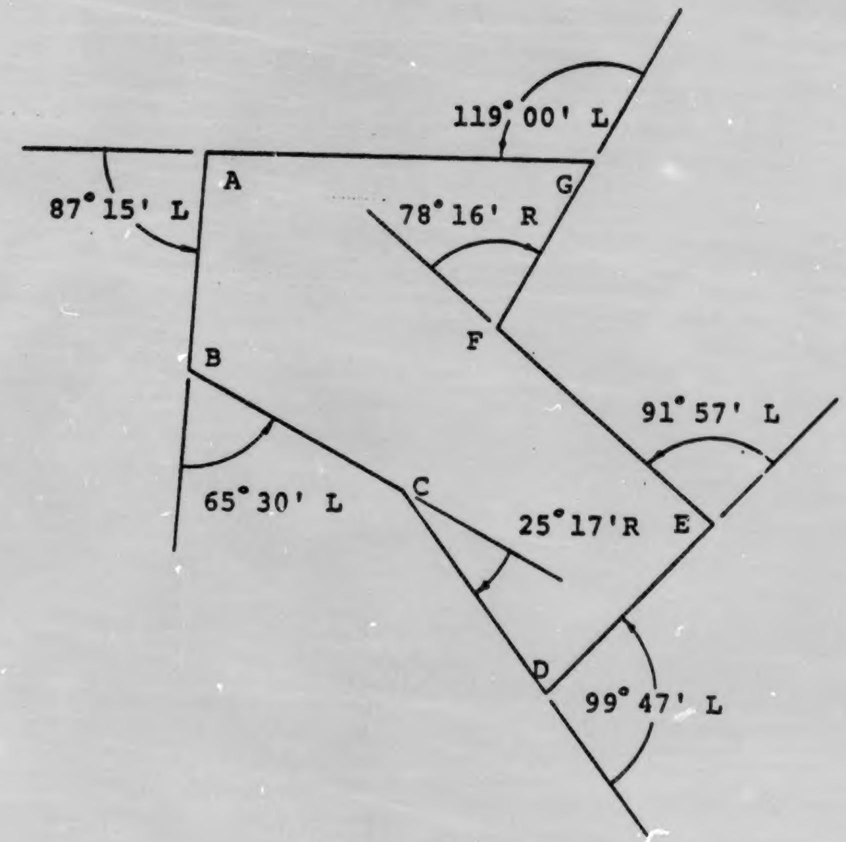

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NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

PROBLEM SHEET

TITLE: Traverse Computations.

OBJECTIVE: The purpose of this problem sheet is to develop your skills in traverse computations.

PROCEDURES:

1. This problem sheet contains a given traverse for which you are to adjust for the linear error of closure as outlined by the instructor. If you experience difficulties raise your hand and the instructor will assist you. Turn in the problem sheet to the instructor by ______________________________.

NAME: ______________________________

BATTALION/CLASS NO: ____________________

DATE: ______________________________

STA	BEARING	DIST.	FUNCTION		LATITUDE		DEPARTURE	
			COS	SIN	N+	S-	E+	W-
A								
	S31° 30' 00"E	50.00						
B								
	N60° 31' 20"E	94.00						
C								
	S36° 05' 20"E	82.00						
D								
	S56° 27' 00"W	80.00						
E								
	S44° 16' 40"E	120.00						
F								
	S56° 58' 40"W	96.00						
G								
	S74° 51' 00"W	95.00						
H								
	N25° 27' 40"W	204.00						
I								
	N42° 44' 40"E	124.00						
A								

STA.	BEARING	DIST.	FUNCTION		LATITUDE		DEPARTURE	
			COS	SIN	N+	S-	E+	W-

LINE	LATITUDE	DEPARTURE	LATITUDE CORR.	DEPARTURE CORR.	ADJUSTED LATITUDE	ADJUSTED DEPARTURE

PROBLEM SHEET EA 410.2

COURSE	ADJ. BEARING	ADJ. DIST.	FUNCTION TAN	FUNCTION SIN	ADJUSTED LATITUDE	ADJUSTED DEPARTURE

[illegible] SHEET EA 410.2

STA.	LATITUDE		DEPARTURE		COORDINATES	
	NORTH	SOUTH	EAST	WEST	Y	X

[illegible] SHEET E. 410.2-

COURSE	LATITUDE		DEPARTURE		DMD	DOUBLE AREA	
	+	-	+	-		+	-

PROBLEM SHEET EA 410.2

COURSE	LATITUDE		DEPARTURE		DPD	DOUBLE AREA	
	+	-	+	-		+	-

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Vertical Control

Average Time: 1 Periods (Class) 6 Periods (Pract)

A. Texts:

1. Engineering Aid 3 & 2, NAVPERS 10634-B

B. References:

1. Engineering Aid 1 & C, NAVEDTRA 10635-B

2. Surveying: Theory and Practice. 5th Edition Davis, Foote and Kelly.

C. Tools, Equipment and Materials:

1. See Annex III.

C. Training Aids and Devices:

1. Films: None

2. Transparencies: None

3. Charts: None

4. Locally Prepared Materials:

a. Information Sheets.

(1) SCBT 410.2 EA IS 1.2.3.1

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acte as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will; while working as a member of a survey part and performing all duties of instrument man, rodman, and notekeeper; establish vertical control for the follow on topics of topographic and road surveying. All surveying procedures will be as specified in Engineering Aid 3 & 2 and this instructor guide. Student results will be to third order with 100 percent accuracy for procedures and calculations.

Criterion Test: Each student will; while working as a member of a survey party and performing all duties of instrument man, rodman, and notekeeper; establish vertic control for the follow on topics of topographic and road surveying. All surveying procedures will be as specifie in Engineering Aid 3 & 2, and this instructor guide. Student results will be to third order with 100 percent accuracy for procedures and calculations.

Homework: Read

OUTLINE OF INSTRUCTION	**INSTRUCTOR ACTIVITY**	**STUDENT ACTIVITY**
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name:		
2. Topic: Vertical Control.		
B. Establish readiness.	I.B. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to establish Vertical Control using trigonometric leveling.	I.D. 1. State information and material necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

II. Presentation

A. Vertical Control.

1. The purpose is to establish bench marks at convenient intervals over the area to serve as:

   a. Points of departure and closure for leveling operations of the topographic parties when locating details.

   b. Reference marks during subsequent construction work.

B. Trigonometric Leveling.

1. The determination of an elevation by use of a vertical angle and a horizontal distance.

   a. The distance map either be scaled from the map or measured directly by stadia or tape.

   b. Usually two or more stations are observed in order to increase the precision of the measurement.

2. It is applicable to field conditions where the horizontal control is established by triangulation and a high degree of accuracy in the measured elevations is not required.

C. Differential leveling (Direct).

1. Establishment of a horizontal plane.

2. Measure vertical distance directly.

3. High degree of accuracy is required.

D. Computing and distributing level error of closure.

1. Level computations.

a. In making level computations, be sure and check on the notes for a level run by verifying the beginning BM, that is by determining that the correct BM was used and its' correct elevation duly recorded.

b. Check the arithemetrical accuracy with which you added backsights and subtracted foresights. The difference between the sum of the foresights taken on BM's or TP's should equal the initial BM or TP and the final BM or TP.

c. You must remember that this checks only the arithmetic. It doesn't indicate anything about how accurately you made the verticcl distance measurements.

2. Precision in leveling.

a. Leveling is carried out in accordance with prescribed "Order of precision".

(1) The instrument used and methods followed must be those which are capable of attaining the specified standard of accuracy.

b. "Order of Precision" - called "First, Second, Third, or even Fourth" order.

(1) The last two are the orders of precision that you will be concerned with usually.

(2) First Order.

(a) Leveling used to establish the main level net,

1. Vertical control for extension of level nets of the same or lower accuracy.

2. Used for mapping, projects, cadastral and local surveys.

(b) Must start and end on a proven bench mark of the same order.

1. New levels must be run between the starting bench mark and at least one other existing bench mark.

2. Must indicate no change in their relative elevations.

(3) Second Order.

(a) Leveling used to subdivide nets of first order leveling.

1. Provides basic control for extension of levels of the same or lower accuracy.

2. Supports mapping projects and local surveys.

(b) Two types - Class I, and Class II.

1. Class I- used in remote areas where the line must be longer than 25 miles.

    a. All lines must start and close on previously established bench marks of first or second order.

    b. New levels must be run between the starting bench mark and at least one other existing mark, indicating no relative elevation change.

2. Class II - used for the development of nets in the more accessible areas.

    a. Same criteria as for Class I except that Class II lines are only run in one direction.

(4) Third order.

(a) Leveling is used to subdivide an area surrounded by first and second order leveling, and to provide elevations for immediate control of cadastral topographic and construction surveys for permanent structures.

(5) Fourth order.

(a) Leveling is used to subdivide an area within a third order network. This method of leveling which is used in connection with location and construction of highways, railroads and most other engineering works that the seabees are concerned with an advanced base projects. However, in practice it is to your advantage if you always try to shoot for a higher degree of accuracy as long as it would not affect the proper progress of the work.

E. Error of Closure.

1. Definition - The difference between the known elevation of the initial BM and the elevation of the same as computed in the level run.

F. Level order of Precision.

1. The precision of a level run is usually prescribed in terms of a maximum error of closure, obtained by multiplying a constant factor by the square root of the length of the run in miles or in kilometers - depending upon which system of measurement is used.

2. The Federal Bureau of Surveying and Mapping specifies certain requirements and maximum closing error, such as shown in Handout SCBT 410.2 EA IS 1.2.3.1

II.F.2. Pass out SCBT 410.2 EA IS 1.2.3.1

OUTLINE OF INSTRUCTION

III. Application

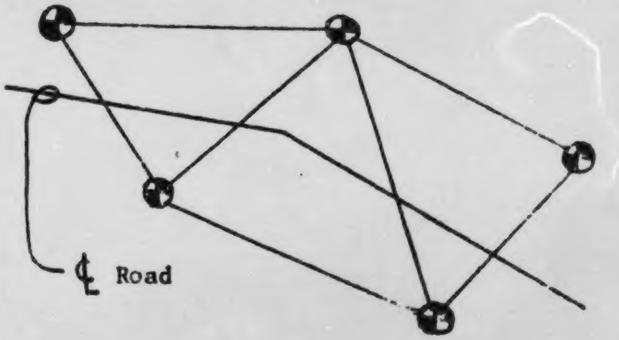
A. Discussion

B. Practical performance.

1. Explain the application activity, cover the following:
    - a. Nature and scope of activity.
        - (1) Run a level loop and establish vertical control.
        - (2) Perform trigonometric leveling.
    - b. Purpose: to give the student practice in establishing vertical control and performing trigonometric leveling.

INSTRUCTOR ACTIVITY	**STUDENT ACTIVITY**
III.A. Questions	III.A. Answers
1. Define trigonometric leveling.	1. The determination of an elevation by use of a vertical angle and a horizontal distance.
2. How is the distance determined in trigonometric leveling?	2. Scaled from a map or measured directly by stadia or tape.
3. When making level computation what should be varified?	3. The beginning BM.
4. What percision is usually used by seabees?	4. Third or fourth.
III.B. This problem should be worked in conjunction with topographic and road surveying problems.	

Suggested Field Layout:



c. Value of activity: Every topographic survey must have vertical control established in order to obtain data in relation to the datum established for a particular survey.

d. Directions for performing activity.

(1) Show the students existing control points.

(2) Form the students in two parties.

(3) Have each party perform the field work on their own.

(4) Turn in field notes upon completion for comparison and grading.

IV. Summary

A. Vertical Control.

B. Trigonometric Leveling.

C. Differential Leveling.

D. Computing and Distributing Level Error of Closure.

E. Error of Closure.

F. Level Order of Precision.

V. Test.

A. None

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

TITLE: Vertical Control

Leveling Accuracy Specifications

REQUIREMENTS	FIRST ORDER	SECOND ORDER Class I	SECOND ORDER Class II	THIRD ORDER	FOURTH ORDER
* Spacing of lines and crosslines ------	72 Km or 40 miles	40-56 km or 25-35 miles	10 km or 6 miles	Not specified	None
Average spacing of permanently marked BMs along lines, not to exceed ------	2 Km or 1 mile	2 Km or 1 mile	2 Km or 1 mile	5 Km or 3 miles	None
Length of sections -	1-2 Km or ½-1 mile	1-2 Km or ½-1 mile	1-2 Km or ½-1 mile	Not specified	None
Check between forward and backward running between fixed elevations or loop closure not to exceed ---------	$4mm \sqrt{K}$ or $0.017 ft. \sqrt{M}$	$8.4mm \sqrt{K}$ or $0.035 ft \sqrt{M}$	$8.4mm \sqrt{K}$ or $0.035 ft \sqrt{M}$	$12mm \sqrt{K}$ or $0.05 ft \sqrt{M}$	0.6 m for lines up to 20 Km. or $0.1 ft \sqrt{M}$

K = the distance in kilometers.
M = the distance in miles.
* In areas outside the U.S. this criteria may be changed to conform with the situation.

○

GURLEY	TRANSIT- #	7321	⊼ EAZ	BROWN	
27 MARCH	1973		Ø EACN	JONE	
			CLEAR	COLD	
ELEV.		REMARKS			
495.0	LEFT BANK OF CREEK				
494.6	RIGHT	" "	"		
495.4	LEFT	" "	"		
496.1	RIGHT	" "	"		

(2 of 5)

LEVEL CIRCUIT

STA	B.S. (+)	H.I.	F.S. (-)	ELEV.	
B.M. 35	6.659	139.822		133.163	
T.P.	4.968	139.319	4.971	134.851	
T.P.	4.508	136.875	7.452	132.367	
T.P.	1.412	132.430	5.857	131.018	
T.P.	7.073	138.2[illegible]2	1.261	131.129	
B.M. 19	24.620		1.785	136.457	
-	21.326		21.326 -	133.163	
	3.294 ←	CHECK	⟶	3.294	

EA 410.2-

⊼ EA2 TODD
ϕ EA3 MILLER

K & E DUMPY
LEVEL 69732

FROM B.M. 35 TO B.M. 19

REMARKS					
APRIL 20, 1973			WEATHER	MILD	
Concrete Monument			WIND	7S	
			TEMP	74°	
Concrete Monument					
			(4 of 5)		

TRIGONOMETRIC LEVELING

	⊼ @ Δ	B.S. A			
	∡'s RIGHT				
	ELEV. 532.4.		H.I. = 4.8		
OBJ.	HOR ∡	ROD INT.	VERT. ∡	HOR. DIST.	DIFF EL
1	30° 10'	6.23	-3° 26'	622	-37.4
2	34°36'	6.55	-3° 18'	654	-37.8
3	30°31'	5.45	-3°54'	543	-37.0
4	38°41'	5.25	-3°58'	523	-36.3

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Topographic Surveying.

Average Time: 3 Periods (Class) 7 Periods (Pract)

Instructional Materials:

A. Texts:

1. Engineering Aid 3 & 2, NAVPERS 10634-B

B. References:

1. Elementary Surveying, Breed & Hosmer.
2. Surveying Theory & Practice, Davis, Foote and Kelly.
3. Surveying Theory & Practices, Tracy.

C. Tools, Equipment and Materials.

1. See Annex III.

D. Training Aids and Devices:

1. Films: None
2. Transparencies: None
3. Charts: None
4. Locally Prepared Materials:

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enablint Objective: The student while working as a member of a survey party performing the duties of instrumentman, rodman, and notekeeper will develop a topographic map by use of statia, trigometric leveling and the controlling point system. Procedures will be as set forth in this instructor's guide and Engineering Aid 3 & 2, NAVPERS 10634-B. All student responses will be 100% accurate for 3 order surveying precision.

Criterion Test: The student while working as a member of a survey party performing the duties of instrumentman, rodman and notekeeper will develop a topographic map from field notes. The student will use stadia, trigometric leveling and the controlling point system for compiling the data used to develop the topographic map. Procedures will be as set forth in this instructor's guide and Engineering Aid 3 & 2 NAVPERS 10634-B. All student responses will be 100% accurate for 3 order surveying precision.

HomeworK; Read

Engineering Aid 3 & 2, NAVPERS 10634-B, Chapter 15, all.

a. Information Sheet.

(1) SCBT 410.2 EA IS 1.2.4.1, Topographic Survey Control Data.

b. Problem Sheets

(1) SCBT 410.2 EA PS 1.2.4.1, Contour Interpolating.

E. Training Aids Equipment.

1. None

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name:		
2. Topic: Topographic Surveys.		
B. Establish readiness.	I.B. Motivate student	
1. Purpose		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of materials being presented.	
1. Value		
a. Pass course		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid		
D. Overview:	I.D. State learning objectives.	
1. You will be able to use stadia, trigometric leveling and the controlling point system for compiling the data used to develop a topographic map.	I.D.1. State information and materials necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

II. Presentation

A. General information

1. Topographic surveying.

a. A survey conducted to gather information from which maps may be made showing the cultural and natural features.

b. Maps are of two types.

(1) Planemetric

(2) Contour

2. Purpose and uses.

a. Designing proposed route locations.

(1) Roads, ditches, sewers, power lines.

b. Calculation volumes.

(1) Barrow pits.

(2) Cut and fill.

c. Designing grades.

(1) All construction (building, drainage, air fields etc.)

3. Basic parts.

a. Features - two types.

(1) Natural (streams, paths, trees, hedges, shorelines etc.)

(2) Artificial features (roads, buildings, fences, power poles, culverts, bridges; man made).

b. Contours.

(1) Definition.

(a) An imaginary line connecting points of equal elevation.

(b) Elevations are determined from mean sea level.

(2) Characteristics of ground slopes and basic land forms.

(a) Uniform slope.

1. Contours appear uniformly spaced on map.

(b) Concave slope.

1. Contours close together at higher elevation gradually becoming farther apart towards lower elevations.

(c) Convex slope.

1. Contours farther apart at higher elevations and becoming closer together towards the bottom of the slope.

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

(d) Basic land forms.

1. Valley or streams.

a. Contours from vee's

/1/ Vee's point uphill or upstream.

(e) Saddle.

1. Has a depression between two summits.

(f) Ridge line.

1. Contours form a "U" shape.

2. Ridge line itself usually shown by a dashed line.

(g) Depression.

1. Shown by a closed contour.

2. Contours have hachures which point to center of depression.

(h) Overhanging cliff.

1. Contours appear to cross each other.

2. Contours are dashed under overhang of cliff.

c. Control

(1) Horizontal.

(a) Base line.

(b) Traverse for primary and secondary points.

(2) Vertical.

(a) Differential leveling.

(b) Stadia.

B. Planning the Survey.

1. The choice of the field method to be used depends upon.

a. Intended use of map.

(1) More refined survey methods for a detailed map than of a general map.

b. The area of the tract.

(1) Control measurements should be more precise for a large tract than for a small tract.

c. Scale of the map.

(1) Large scale (1" = 100' or less).

(2) Intermediate scale (1" = 100' to 1" = 1000').

(3) Small scale (1" = 1000' or more).

NOTE: Do not discuss selection at this time.

d. Contour interval

(1) The smaller the contour interval the more refined should be the field method.

NOTE: Do not discuss selection at this point.

C. General Field Methods.

1. The instruments used are transit, plane table, engineer level, hand level and clinometer. But the transit has advantages over plane table, so we will work only with it.

2. Control is established.

3. Details are located in four general systems.

a. Cross-profile.

(1) Used on route surveys or flat ground.

(2) Short lines transverse to the main traverse.

(3) Distance off traverse measured with tape.

(4) Elevation by direct leveling, often with a hand level.

(5) Used for maps of intermediate scale surveys.

b. Checker board.

(1) Used where the scale is large and the tract is wooded or the topography is smooth.

(2) Tract is divided into squares or rectangles with stakes at the corners.

(3) Elevation is determined at the corners and intermediate points of changes in slope.

(4) Elevation is usually by direct leveling.

b. Trace contour.

(1) Contours are traced out on the ground.

(2) Points occupied by rod are located by radiation with transit or plane table.

(3) Frequently the engineer's level is employed as an auxiliary instrument.

(4) Used on large scale surveys where required accuracy is high and the ground is somewhat irregular in form.

c. Controlling point.

(1) Ground points form at irregular system along ridge and valley lines and at other critical features.

(2) Ground points located by radiation or intersection with transit or plane table.

(3) Elevations are commonly determined by trigomometric leveling or sometimes by direct leveling.

(4) Small scale surveys.

e. A combination of systems can be used for different features within one map area.

D. Location of Details.

1. Angle and distance from transit station.

a. Distance usually is by stadia.

b. Most widely used method.

2. Angles from two transit stations.

a. Useful in locating distant or inaccessible objects.

b. Object must be seen from two or more transit stations.

3. Distance from two stations.

a. Work expedited when traverse are staked out every 100 feet.

b. Best used for details relatively close to the traverse line.

4. Angle from one station and distance from another.

a. Used occasionally where advantageous.

b. Generally only when an obstacle is between the transit station and the object (river).

5. Perpendicular offset from the control line.

   a. For location of irregular or curved boundaries, streams and roads that closely parallel the transit lines.

   b. Used also on route surveys as a method of location.

6. Accuracy of location.

   a. Dependent upon

      (1) Ultimate use of map.

      (2) Map scale.

      (3) Precision in plotting.

E. Accuracy.

1. Depends upon weather.

2. Use of map.

3. Selection of proper equipment.

4. Procedure.

F. Topographic Maps - Shows by the use of suitable symbols (1) the configuration of the earth's surface called relief (2) natural features such as trees and (3) the physical changes wrought upon the earth's surfaces by the work of man.

1. The distinguishing characteristics of a topographic map, as compared to other maps is the representation of the terrestrial relief.

2. Uses

   a. In design by engineers, geologists, economists, surveyors and others.

   b. The preparation of general topographic maps is largely in the hands of governmental organizations.

G. Representation of Relief.

1. Shading - pictorial relief.

   a. A method of showing terrestrial relief roughly in plan.

   b. Vertical point shows with parallel light rays.

   c. Causes shadows to lie upon the less illuminated areas.

   d. Used best where relief is high and steep.

   e. Used often with hachures of contour lines.

2. Hachures.

   a. Shows relief more definitely but less legibly than shading.

   b. Consists of rows of short lines nearly parallel.

3. Contour and Contour lines.

   a. Defined-is an imaginary line of constant elevation on the ground surface.

(1) Distance between contour lines is the contour interval.

b. Characteristics.

(1) The horizontal distance between contour lines is inversely proportional to the slope.

(2) Uniform slopes/spaced uniformly.

(3) Along plane surfaces such as Railroad Cuts and Fills the contours are straight and parallel.

(4) Contour lines represent level lines, they are perpendicular to the lines of steepest slope, ridge, and valley lines where they cross.

(5) All contour lines close upon themselves.

(a) Depressions indicated by Hachures.

(b) Summits by elevation.

(6) Contour lines cannot cross or merge except in the rare case of vertical surfaces.

(a) Bridge abutments.

(b) Overhanging ground.

(c) Cliffs.

(d) Caves.

(7) A single line cannot lie between two contour lines of higher or lower elevation.

c. Contour Interval.

(1) Factors affecting selection.

(a) Desired accuracy of elevation to be read from map.

(b) Characteristics of Terrain to be mapped.

(2) Contours are generally not spaced closer than 20 or 30 lines to the inch.

(3) Other features must not be obscured by contours too close together.

d. Interpolation - The process of spacing the contour lines proportionately between plotted points.

(1) A uniform slope must be assumed between points.

(2) Methods of interpolation.

(a) Estimation

1. Contours not located accurately but only by eye between points.

2. Method most commonly used on intermediate and small scale maps.

(b) Computation

1. By proportioning.

2. Most accurate method.

3. Method is very laborious and takes time.

H. Representative of Features.

1. Natural - Those that exist before man, trees, streams, lake, hills, and valleys.

a. Symbols in the text should be covered with tips on drawing.

2. Artificial - The physical changes wrought upon the earth's surface by the works of man, such as houses, roads, canals, and cultivation.

a. Symbols in the text should be covered with tops on drawing.

I. Drawing the map.

1. Sheet format as per MIL STDS.

2. Scale.

a. Accuracy - Based on purpose of survey if plotting error is known.

Error = 1/40

Scale distance error = 10'

Map scale 1" = 400 Ft.

b. Clarify with which features can be shown.

c. Cost.

d. Correlation of data with related maps.

e. Desired size of Map Sheet.

f. Physical factors/number and characteristics of features to be shown.

g. Nature of terrain.

h. Contour interval (20 to 30 max. per inch).

3. Meridian arrows.

a. Direction indicated by a needle or feathered arrow pointing North of sufficient length to be transferred with reasonable accuracy.

b. True meridian is usually represented by an arrow with full head.

c. When both are shown, the angle between them should be indicated.

4. Lettering.

a. Lettering shall conform to MIL-STD 100A.

b. Lettering will be placed so that it can be read from the bottom of the sheet.

c. Lower case lettering may be used on topographic maps, but in general only use it on lesser important items such as the following:

(1) Control point description.

(2) Names of property owners, of surrounding land or discription of the land, orchards, swamps, etc.

(3) Description of fence lines, stream centerlines, trails, etc.

d. Upper case lettering should be used in conjunction with lower case, but only for the important items such as:

(1) Names of woods.

(2) Names of rivers.

(3) Title of maps.

(4) Any object of major importance.

5. Line weight.

a. Thick lines - Border, Legend Block, Title Block.

b. Medium lines.

(1) Object outlines (Bldgs, Roads etc.)

(2) Index contours and lettering of elevations.

(3) Control.

(4) North arrow.

c. Thin lines.

(1) Intermediate contours.

(2) Edge of wooded areas.

(3) Centerline of streams.

(4) Edge of dirt roads and paths.

6. Plotting.

a. All plotting should be done from control points with a protractor.

b. Label each point or station as it is plotted.

7. Steps in drawing a map.

a. Usually done on a rough field sheet first, then traced.

b. Plotting the horizontal control or skeleton of the map.

(1) Label control points including elevation on points.

c. Plotting the location of detail (ground points).

(1) Label points, including elevations on points.

d. Construction of contour lines.

(1) Label index contours.

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

8. Procedures for tracing from the field sheet.

   a. Make sure all lines drawn on the field sheet are dark enough to be seen through the tracing paper.

   b. Tape field sheet down on the drawing board and determine MIL STD tracing size to best fit field sheet.

   c. Place tracing over field sheet and position, keeping North arrow pointing up the sheet and insuring that enough room is left for a legend and title block in the lower right hand corner.

   d. After taping down on drawing board, and using a 6H pencil, layout or trace the following:

      (1) Border, title block, legend and North arrow.

      (2) Trace all details and controls.

      (3) Rule guide lines for lettering and carefully plan what and where information will be placed on the tracing.

      (4) Now tracing is ready for finishing details.

9. Miscellaneous.

   a. The following are some tips to keep in mind while drawing maps.

      (1) Simplicity.

         (a) A cluttered map is extremely difficult to read.

b. Insure that all information on a map has only one interpretation.

c. All symbols on a map should have an unmistakable similarity to objects being represented.

d. A graphic scale should be put on the map.

e. A map must be inked for resistance against usage and for clarity in prints.

10. Finsihed details.

a. After plotting is complete and checked for accuracy it is ready for:

(1) Border lines.

(2) Title block.

(3) North arrow,

(4) Legend.

(5) Contour interval and control information.

11. Checking for accuracy.

a. Can be checked both in plan and elevation.

b. Compare horizontal dimensions on distances scaled from the map and distance measured on the ground between corresponding points.

c. Compare elevations by:

(1) Elevations determined by field levels with those taken from a map for selected points.

OUTLINE OF INSTRUCTION — INSTRUCTOR ACTIVITY — STUDENT ACTIVITY

(2) Profiles from field notes compared against a profile plotted from the map.

III. Application.

A. Discussion

III.A. Questions	III.A. Answers
1. What two types of maps are discussed?	1. Planemetric and contour.
2. What are the basic parts of a topographic survey?	2. Features, contours and control
3. A "U" shaped contour would indicate what?	3. Ridge line.
4. What four items control the choice of a field method of survey for a topographic survey?	4. Use of map, area of tract, scale and contour interval.
5. What system is commonly used for small scale maps?	5. Controlling point.
6. Name three methods used to show relief.	6. Shading, Hachures and Contour lines.
7. Can contour lines cross?	7. No.
8. What is contour interval?	8. Distance between contour lines.
9. Name two methods of interpolation.	9. Estimation and computation.
10. What weight line is used for contour line?	10. Thin lines.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
	LL. What is a field sheet?	11. First drawing used in developing a topographic map.
IV. Summary.		
A. General information.		
1. Topographic surveying.		
2. Purpose and use.		
3. Basic parts.		
B. Planning the survey.		
1. Choice of field method.		
C. General Field Method.		
1. Instruments used.		
2. Control.		
3. Four general systems.		
D. Location of details.		
E. Accuracy.		
F. Topographic maps.		
1. Definition		
2. Uses.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

G. Representation of Relief.

  1. Shading.
  2. Hachures.
  3. Contour and contour lines.

H. Representation of Features.

  1. Natural.
  2. Artificial.

I. Drawing the Map.

  1. Format.
  2. Scale.
  3. Meridian.
  4. Lettering.
  5. Line weight.
  6. Plotting.
  7. Steps in Drawing Map.
  8. Procedures for tracing from field sheet.
  9. Miscellaneous.
  10. Finishing details.
  11. Checking for accuracy.

V. Test: None

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NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

PROBLEM SHEET

TOPIC: Contour Interpolation.

OBJECTIVES: This problem sheet has been prepared to give you needed practice in contour interpolation. Successful completion of this problem sheet is a good indication that you are capable of drawing the contours for a topographic map.

PROCEDURES:

1. Use the problem worked in class as a guide. All needed information for the solution will be given. Show all work on problem sheet.

2. Draw 5.0 feet contours (5, 10, 15, 20, etc.) by interpolating elevations shown on the plot. (Figures at the corners represent elevations.)

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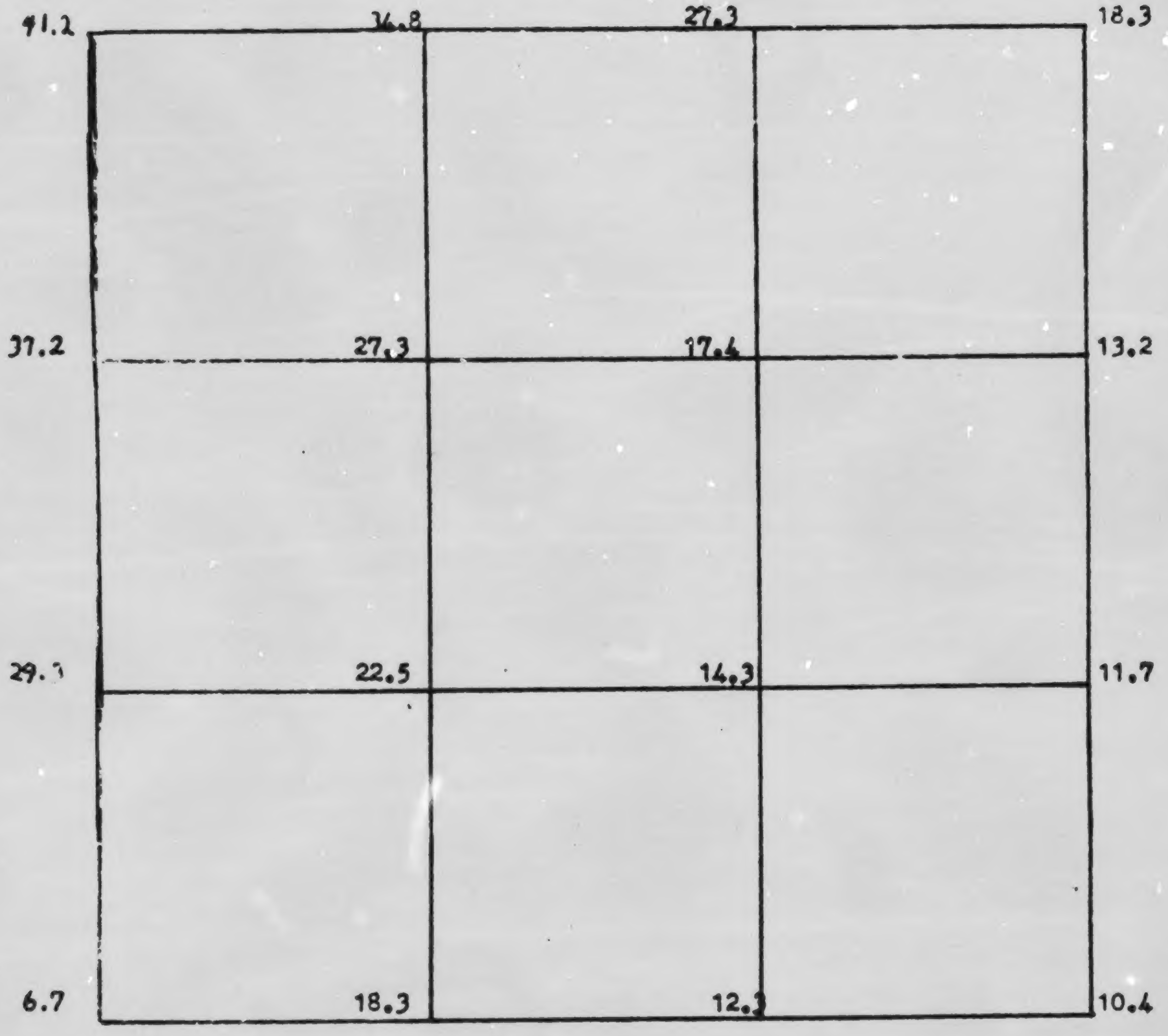
PROBLEM SHEET EA 410.

SCBT 410.2 EA PS 1.2.4.1

[illegible] 5 FT. CONTOURS (5, 10, 15, 20, ETC.)

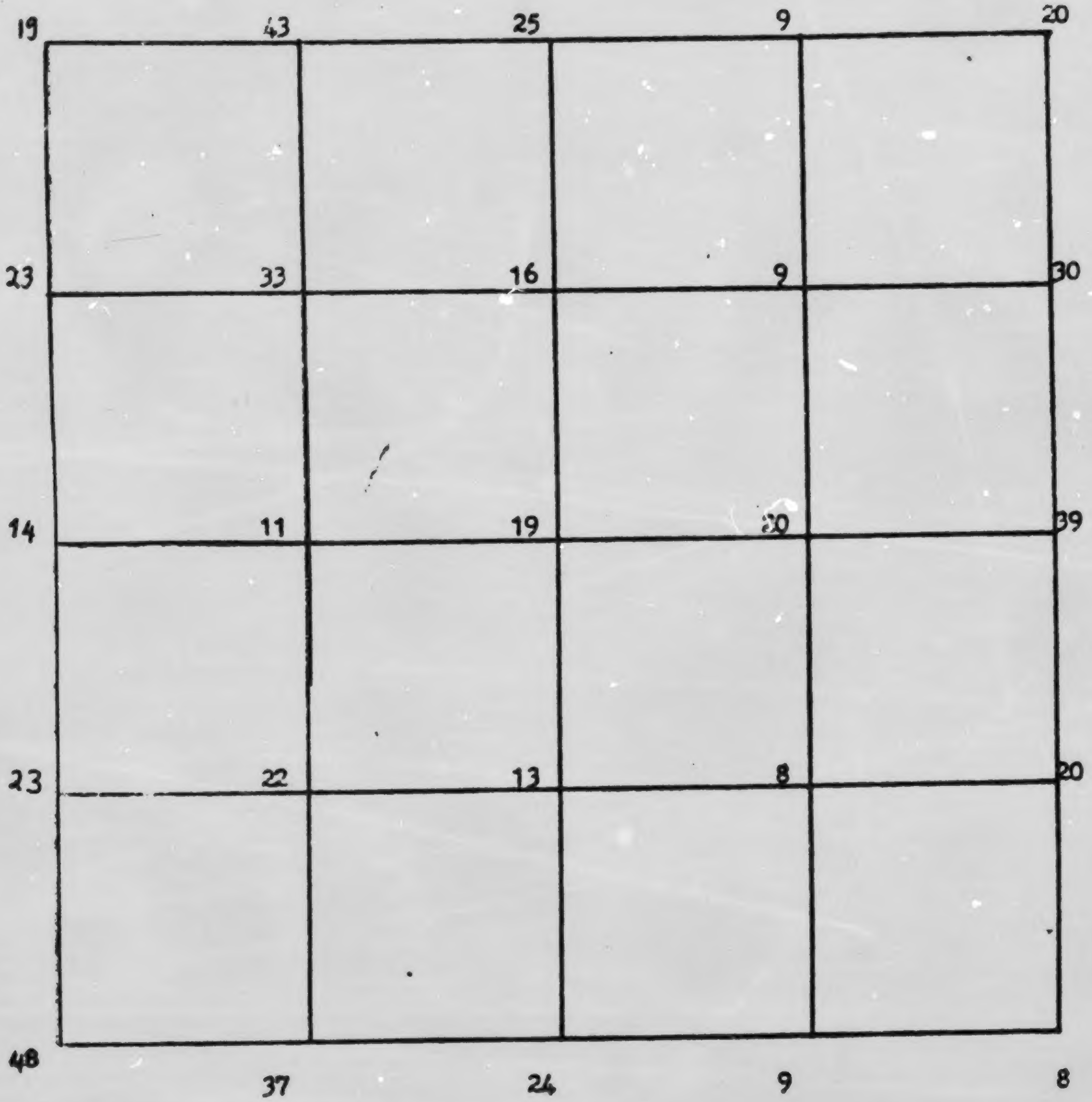


(2 of 3)

[illegible] SHEET EA 410,.

SCBT 410.2 EA PS 1.2.4.1

[illegible] FT. CONTOURS (5, 10, 15, 20, ETC.)



19	43	25	9	20
23	33	16	9	30
14	11	19	20	39
23	22	13	8	20
48	37	24	9	8

(3 of 3)

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Road Surveying

Average Time: 2 Periods (Class) 7 Periods (Pract)

Instructional Materials:

A. Texts:

   1. Engineering Aid 3 & 2, NAVPERS 10634-B

B. References:

   1. Surveying Theory & Practice

   2. Engineering Surveys.

C. Tools, Equipment and Materials:

   1. See Annex III.

D. Training Aids & Devices:

   1. None.

E. Training Aids Equipment:

   1. None

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student willhave acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topi objectives.

Enabling Objectives: Upon completion of this topic each student will have performed the procedures involved in road surveying for location and layout. Each student wil act as chairman, rodman, notekeeper and instrumentman in meeting the requirements of this topic. All procedures will be as outlined in this topic and Engineering Aid 3 & NAVPERS 10634-B, Chapter 16. All survey work will be to third order accuracy, all profiles to the nearest 1/10 foot and all cross sections to the nearest 1/10 foot. Al math computations will be 100 percent accurate as checked

Criterion Test: Each student will act as chairman, instr mentman, rodman and notekeeper in a survey party when per forming the survey procedures necessary to establish two straight sections of road at least 500 feet in length and meeting at a 30 degree horizontal angle at their intersection, including cross section and plan profile drawing The road will have a flat gradiant over its entire course except the last 200 feet of one section which will have a elevation change of 16 feet with a constant grade. All survey work will be to third order accuracy, all profiles to the 1/10 foot, all cross sections to the nearest 1/10 foot and all math computations will be 100 percent accura as checked.

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Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B, Chapter 16. pp. 503-531.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Road Surveying		
B. Establish readiness.	I.A. Motivate student.	
1. Purpose		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview.	I.D. State learning objectives.	
1. You will be able to perform the procedures involved in road surveys.	I.D.1. State information and materials necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

II. Presentation.

A. Sequence of operation involved in engineering surveys.

1. Reconnaissance Surveys.

a. First evaluation of project site.

(1) Stude made of available maps, drawings and aerial photographs.

(2) Talk with personnel familiar with the project site.

(3) Possible field trip to reconnoiter project site.

b. Equipment for reconnaissance should include the following:

(1) Compass, barometer, abney hand level, tapes, binoculars and cameras.

c. Field reconnaissance should include the following:

(1) Sketches of route or area.

(2) Written report on the route or area.

(a) Includes.

1. Clearing and grubbing of the site.

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

2. Depth of streams and rivers, etc.

3. Description and sizes of marsh areas or other obstacles which may be encountered.

4. Any grade or alignment problems.

5. Possible effects of snow, rainfall, land slide, etc., on the site.

6. Soil conditions.

7. Any discrepancies noted on maps or aerial photographs.

8. Photographs of the site and any control which was noted.

9. The availability of local material, labor, equipment and transportation.

10. Any other information which you think may be useful in the development of the site.

11. Any recommendations you may have.

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

2. Preliminary survey.
   a. Detailed study of the area or route.
   b. Information gathered to determine feasibility of constructing project on site selected during reconnaissance.

3. Location survey.
   a. Establish the approved layout in the field (center line).
   b. Taken from data collected during preliminary phase.

4. Construction/layout survey.
   a. Provide final alignment, grade and final location to guide construction.
      (1) Place clearing and grubbing stakes.
      (2) Layout project.
      (3) Grade stakes.
      (4) Slope stakes.
      (5) Finish elevation (blue top).
   b. Keep records and make checks as construction progresses.
   c. Data obtained can be used for as-builts survey.

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

B. Road surveying - eight steps.

1. Horizontal control.

a. Use a traverse (open or closed).

(1) Record all information.

b. Set all traverse points and references.

2. Vertical control.

a. Establish elevations on all traverse stations and references.

(1) Record information.

3. Center line control.

a. Stakes set at all 100 foot stations and all changes of direction.

b. Stationing.

(1) Full station (1 + 00) etc.

(2) Plus station (1 + 50) etc.

(a) Any point other than full station.

c. Record all information.

-. Profile and cross-sections.

a. Can be run together or separately.

b. Profiles.

  (1) Procedure same as open level circuit.

  (2) Ground elevations are taken at all center line stations (full and plus).

  (3) Center line elevations are taken at all major breaks in the ground.

  (4) Generally elevations are taken to the nearest:

    (a) Tenths (0.1 ft.) on existing ground.

    (b) Hundredths (0.01 ft.) is used for finished surfaces.

    (c) Hundredths (0.01 ft.) is used for all T.P.'s.

  (5) Record information.

    (a) Recorded as level note for differential level circuit.

    (b) Right hand page to be used for description information.

c. Cross sectioning.

  (1) A cross sectional view is made of each full and of selected plus stations.

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

(a) Used in stake-out and earth-moving calculations.

(2) Run cross-section levels.

(a) Take ground shots at breaks in the ground, at right angles to the center line, no further out than the edge of right of way.

1. Unless side slope should go out further.

(b) Hand level or engineer's level can be used.

(c) Cross-sections are taken to the nearest (o.1) of a foot.

(d) Record both the rod reading and the dist. out from the center line.

(e) This is done both on the right and left of the center line.

(f) The job and the terrain will determine how far out from the centerline you will go.

(g) This is repeated on each station.

(3) Record information.

(a) Left hand page is kept as a normal circuit.

(b) Right hand page is as follows.

  1. Center line of page represents the center line of the route.

(c) Distances, elevations and rod readings to the right of the center line are written on the right side of this page: left on left side bottom to top of page.

(d) For each shot three numbers are written.

  1. Top number - distance from center line.

  2. Middle number - rod reading.

  3. Bottom number - ground elevation of that point.

  4. Center line has same set up as right and left sides.

5. Paper location.

  a. Plot all information from steps 1, 2, 3 and 4 of (d) above.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
b. Draw horizontal control field sheet.		
c. Draw center line field sheet.		
d. Draw profile on profile paper alone with center line and horizontal control traced from field sheet.		
(1) Profile paper.	II.B.5.d.(1) Show samples.	
(a) Specially ruled paper.		
(b) Rulings are printed in colored ink (green or orange).		
(c) Common type profile paper has a heavy line 2 1/2 inches apart and lighter lines 1/4 inch apart.		
(d) Generally comes in rolls.		
(e) Plan profile paper.		
(2) Method for plotting profiles		
(a) Secure paper to board or table, leave paper attached to roll.		
(b) Set up scale.		
1. Horizontal scale along bottom from left to right.		

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

2. Vertical scale - bottom to top.

3. Vertical scale is exaggerated in relation to the horizontal scale to give a better picture of the ground VERTICAL 1" = 10', HORIZONTAL 1" = 100' to 300'.

(c) To plot an elevation for a station from the station number at the bottom of the paper move up until it intersects the line across from the elevation that is to be plotted. Make a point on the paper.

(d) Connect the points with a line.

(e) Finished plot is now a picture of existing ground along center line of road.

1. A graphic display of the shape of the center line elevation.

e. Draw cross-sections.

II.B.5.e.(1) Show samples of previous student work.

(1) Plotted on cross-section paper.

(a) Cross-section paper.

1. Vertical and horizontal lines are heavy lines 1 inch apart with lighter lines, 1/20, 1/10, 1/8 or 1/4 inch apart.

2. Comes in rolls or sheets.

(2) Scales generally used.

(a) Cross-section 1 inch equals 10'.

(3) Method for plotting cross-sections.

(a) Single horizontal distance is measured from the center line, pick a vertical line and mark its center line.

(b) Place the first cross-section near the bottom and to the left of the paper.

(c) Proceed in plotting the same as profiles.

(4) As one cross-section is completed, move directly above using the same center line and plot the next station.

f. Draw in roadway design.

(1) Roadway elevations from designer.

(2) Side slopes.

(a) The rate of side slope stated in terms of the number of units measured horizontally to one unit measured vertically.

(b) Thus, a 2:1 slope would have a horizontal distance (run) of two feet and a vertical distance (rise) of one foot.

(c) Common side slopes.

1. Ordinary earth = 1 1/2:1.
2. Coarse gravel = 1:1.
3. Loose rock = 1/2:1.
4. Solid rock = 1/4:1.
5. Sand and clay = 2 or 3:1.

6. Grade and earthwork calculations.

a. Usually done by the grade foreman of the planning and estimating staff (will be discussed in detail in Topic 1.2.5 "Grade and Earthwork Calculations").

b. They are used for.

(1) Degree of vertical curve.

(2) Balance cuts and fills.

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

(3) Provide for drainage.

(4) Limit haul material.

(5) Rough paving estimates.

7. Final location survey.

a. Once paper location is determined, the process to locate final alignment including staking out centerline, curves, final profile and cross-sections for grade and slope stakes.

(1) Staking is carried out in field for construction survey.

8. "As-built" surveys.

a. Once construction is complete it is again resurveyed and any changes made during construction are recorded.

III. Application.

A. Discussion.

B. Practical Performance.

IV. Summary.

A. Road surveying - eight steps.

1. Horizontal control.

2. Vertical control.

III.A. Questions.	III.A. Answers.
1. In vertical control where would you establish elevations?	1. All traverse stations
2. What is a plus station?	2. Any point other than a full station.
3. Generally elevations are read to the nearest what in profile leveling for existing ground?	3. 0.1 feet.
4. What is the top number on a set of cross-section notes?	4. Distance from center line.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
3. Center line control.	III.B. Explain problem outlined in the criterion test to students. Visit site for prior recon. Have students perform road survey steps 1, 2, 3, 4 and 5.	III.B. Perform required survey work.
4. Profile and cross-sections.		
5. Paper location.		
6. Grade and earthwork calculations.		
7. Final location survey.		
8. "As-built" surveys.		
V. Test.		
A. See criterion test.		

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Road Surveys.

Average Time: 2 Periods (Class) 4 Periods (Pract)

Instructional Materials:

A. Texts:

1. Engineering Aid 3 & 2, NAVPERS 10634-B.

B. References:

1. Engineering Aid 1 & C, NAVEDTRA 10635-B.

C. Tools, Equipment and Materials:

1. See Annex III.

D. Training Aids and Devices:

1. Films: None

2. Transparencies: None.

3. Charts: None

4. Locally Prepared Materials:

a. Information Sheets.

(1) SCBT 410.2 EA IS 1.2.6.1

Being Developed

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have act as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Co ditions and standards are as set forth in individual to objectives.

Enabling Objectives: Upon completion of this topic eac student while working as a member of a survey party and performing the duties of instrumentman, rodman, notekee and chainman will stake out the road designed in topic 1.2.5 "Road Surveying". Students will follow the procedures established by this instructor's guide and Engineering Aid 3 & 2 for setting centerline and relate grade stakes and slope stakes. All student field work will be 100% for procedures and to third order for surveying accuracy.

Criterion Test: Each student will set centerline and r lated grade and slope stakes in staking out the road de signed during the performance of topic 1.2.5, "Road Surveying". Each student will work as a member of a su vey party performing the duties of instrumentman, rodma notekeeper and chainman. Student procedures will be as established in this instructor's guide and Engineering Aid 3 & 2. All student field work will be 100% for pro cedures and to third order for surveying accuracy.

Homework: Read

Engineering Aid 3 & 2, Chapter 16, pp. 523 thru 531.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Road Surveys.		
B. Establish readiness.	I.B. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aids.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to, when provided with a design, stake out a road.	I.D.1. State information and materials necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

II. Presentation.

A. Road Stakes.

1. Centerline grade stakes.

a. Grade stakes are used as a guide for construction.

b. Grade stakes are set at grade or some known distance above or below the desired grade.

c. Information is marked on the stake, either by written terms or symbols.

(1) Symbols used:

(a) C = CUT $\underline{\vee}$

(b) F = FILL $\overline{\wedge}$

d. Rough grading.

(1) The final grade has been computed and placed on the profile.

(2) From the profile the centerline cut of fill can be computed.

(3) Cut of Fill is recorded and marked on the back of the centerline stakes, to the nearest 0.1 foot.

(4) Rough grade stakes are out on each side of centerline and edge of shoulder.

e. Final (finish) Grade (Hubs).

(1) Hub stakes are driven to have top of stake to grade.

(2) The tops of hub stakes are colored with keel, (blue tops) (Red tops).

(3) One color for sub-grade and another for finished grade.

f. Offset grade stakes.

(1) Grade stakes can be offset so as to not be disturbed by construction.

g. Setting grade stakes.

(1) Set level near BM.

(2) BS and compute HI.

(3) Compute grade rod for elevation desired.

(a) HI above grade, grade rod is HI minus elevation.

(b) HI below grade, grade rod is elevation minus HI.

(4) Cut and Fill.

(a) Cut at any point: grade rod minus ground rod.

(b) Fill at any point equals:

1. Ground rod - grade rod (if HI is above grade).

2. Ground rod + grade rod (if HI is below grade.)

1⁄3

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

(5) Explain on chalkboard.

h. Note keeping (grade stakes).

(1) Notes are kept in same manner as level notes, except far right column on left page which is used for C or F (cut or fill). Sta, BS, HI, FS, Elev, Grade, C/F.

2. Slope stakes.

a. Definition.

(1) Stakes that define the extremities of the cut or fill.

(2) Computing distance from centerline.

(a) Three things must be known:

1. Width of proposed roadbed (w).

2. Side slope ratio (S).

1 3. Vertical distance above grade at the slope stake (H).

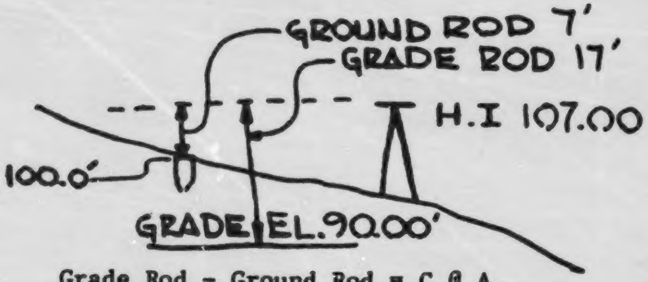
(b) Expressed in formula:

$$d = \frac{W}{2} + hs$$

b. Setting slope stakes.

(1) Stake is set by trial method, since existing ground elevation at right angles to centerline are not known.

(a) Instructor explain on chalkboard the following example:



Grade Rod - Ground Rod = C @ A, 17' - 7' = 10' cut @ "A" 10 X 1.5 (1 1/2 to 1 slope) = 15' + 20' (1/2 road bed) = 35' out for slope stake "B". This would be where slope stake would go if ground rod at "A" and "B" were the same but; ground rod @ "A" = 7'.

Ground rod @ "A" = 4; ∴ 7' - 4' = 3' x 1.5' = 4.5' further out.

Ground rod @ "C" = 3'; ∴ 4' - 3' = 1' x 1.5' = 1.5' further out.

Ground rod @ "D" = 2.9' which only misses 3' by 0.1'

(2) Notekeeping (slope stakes).

(a) Column #1 Stationing.

(b) Column #2 Centerline elevation.

(c) Column #3 Proposed grade elevation.

(d) Column #4 Distance from centerline above cut of fill of left slope stake.

(e) Column #5 Cut of fill to grade at centerline.

(f) Column #6 Distance from centerline above cut or fill of right slope stake.

(g) Example.

Slope Stake

CL Grade Base 40' - S.S. 1 1/2 to 1

STA	ELEV	ELEV	CL	CL	R
0+50	151.5	150.7	29.0 -6.0	-0.8	22.4 -1.6

(3) Stake marking.

(a) Station number on back.

(b) Cut or fill over distance out on front.

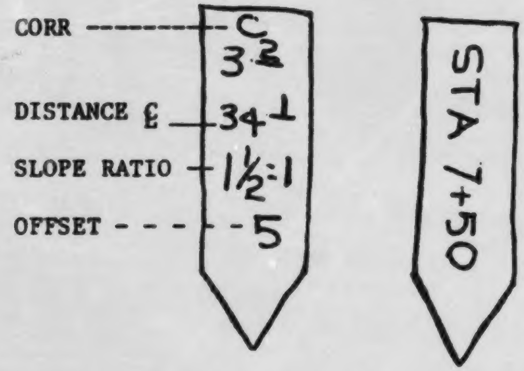
(c) Show side slope ratio.

(4) Stake driving.

(a) Drive stakes so they:

1. Slope toward center (inward) for cuts.

2. Slope away from centerline (outward) for fills.



OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
III. Application.		
A. Discussion.	III.A. Questions	III.A. Answers
	1. What is a grade stake?	1. It demotes cut or fill from a known datum to a desired elevation.
	2. What is a blue top?	2. Finish grade stake.
	3. What is a slope stake?	3. Defines limits of cut or fill.
B. Practical Performance.	III.B. Read and explain the Criterion Test. Have students perform same.	III.B. Perform the requirements of the Criterion Test.
1. Students will stake out road designed in Topic 1.2.5 "Road Surveying".		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

IV. Summary.

   A. Road stakes.

      1. Centerline stakes.

      2. Slope stakes.

V. Test.

   A. None.

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043

TOPOGRAPHIC SURVEY CONTROL DATA
(APPROXIMATE VALUES)

SCALE OF MAP	KIND OF CONTROL	TRAVERSE			LEVELS	
		LENGTH OF TRAVERSE, MILES	MAX. ERROR OF ANGLES	MAX. LINEAR ERROR OF CLOSURE	LENGTH OF CIRCUIT MILES	MAX. ERROR OF CLOSURE
SMALL	PRIMARY	50 to 500	2"	$\frac{1}{25{,}000}$	50 to 500	0.017 Ft. X $\sqrt{\text{Miles}}$
	SECONDARY	25 to 200	5"	$\frac{1}{10{,}000}$		0.035 Ft. X $\sqrt{\text{Miles}}$
	TERTIARY	10 to 100	30"	$\frac{1}{5{,}000}$	10 to 100	0.05 Ft. X $\sqrt{\text{Miles}}$
	QUATERNARY	1 to 2	2' or Compass	$\frac{1}{1{,}000}$	1 to 10	0.1 to 0.5 Ft. X $\sqrt{\text{Miles}}$
INTERMEDIATE	PRIMARY	1 to 20	10" to 1'	1/1,000 to 1/5,000	1 to 25	0.05 to 0.3 Ft X $\sqrt{\text{Miles}}$
	SECONDARY	1 to 5	30" to 3'	1/500 to 1/2,500	1 to 5	0.1 to 0.5 Ft. X $\sqrt{\text{Miles}}$
LARGE	PRIMARY	1 to 5	30" to 1'	1/5,000 to 1/20,000	1 to 10	0.05 to 0.1 Ft X $\sqrt{\text{Miles}}$
	SECONDARY	½ to 3	30" to 2'	1/1,000 to 1/5,000	1 to 3	0.05 to 0.1 Ft X $\sqrt{\text{Miles}}$

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Title: Horizontal Curves.

Average Time: 3 Periods (Class) 9 Periods (Pract)

Instructional Materials:

A. Texts:

1. Engineering Aid 3 & 2, NAVPERS 10634-B.

B. References:

1. Surveying Theory & Practice, 5th Edition, Davis, Foote & Kelly.

2. Surveying, Legault, McMaster and Marlette.

C. Tools, Equipment and Materials:

1. See Annex III.

D. Training Aids and Devices:

1. Films: None.

2. Transparencies: None.

3. Charts: None

4. Locally Prepared Materials:

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will be able to design a horizontal curve and set up field notes for use by a survey party; consisting of four (4) students who will perform all the duties of an instrumentman, chainman, and notekeeper; in laying out the curve on the road centerline established by Topics 1.2.5 "Road Surveying", and 1.2.6 "Road Stakes". Design and layout procedures and computations will be as specified in this instructor's guide and Engineering Aid 3 & 2 NAVPERS, 10634-B. Students will be 100 percent correct for all computations and procedures for designing the curve and in setting up the field notes; all field work performed by the survey party will be to 3rd order precision.

Criterion Test: Each student will design a horizontal curve and set up field notes for use by a survey party consisting of four (4) students performing all the dutie of instrumentman, chainman and notekeeper in laying out the curve on the road centerline established by Topics 1.2.5 "Road Surveying", and 1.2.6 "Road Stakes". Design and layout procedures and computations shall be as specified in this instructor's guide and Engineering Aid 3 & 2

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

a. Information Sheets.

(1) SCBT 410.2 EA IS 1.2.7.1, Horizontal Curve.

(2) SCBT 410.2 EA IS 1.2.7.2, Arc and Chord Definition.

(3) SCBT 410.2 EA IS 1.2.7.3, Horizontal Curve Formulas.

(4) SCBT 410.2 EA IS 1.2.7.4, Sample Note Format (Horizontal Curve).

(5) SCBT 410.2 EA IS 1.2.7.5, Curve Computation and Correction Tables.

E. Training Aids Equipment:

1. None.

NAVPERS 10634-B. Students will be 100 percent correct for all computations and procedures for designing the curve and setting up of the field notes, all field work performed by the survey party will be to 3rd order precision.

Homework: Read

Engineering Aid 3 & 2, Chapter 16, pp. 505 thru 512.

OUTLINE OF INSTRUCTION	**INSTRUCTOR ACTIVITY**	**STUDENT ACTIVITY**
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Horizontal Curves.		
B. Establish readiness.	I.A. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to compute and layout in the field horizontal curves.	I.D.1. State information and materials necessary to guide students.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

II. Presentation.

A. Horizontal Curves - A circular arc connecting two tangents which show a change in direction.

1. Definitions.

a. P.I. - Point of intersection of the two tangents.

b. P.C. - Point of change from tangent to curve or the beginning of the circular arc.

(1) Called point of curvature.

c. P.T. - Point of change from curve to tangent or end of circular arc.

(1) Called point of tangency.

d. A. - Central angle.

(1) Angle between the radii subtending the total arc.

e. I. - Angle of intersection.

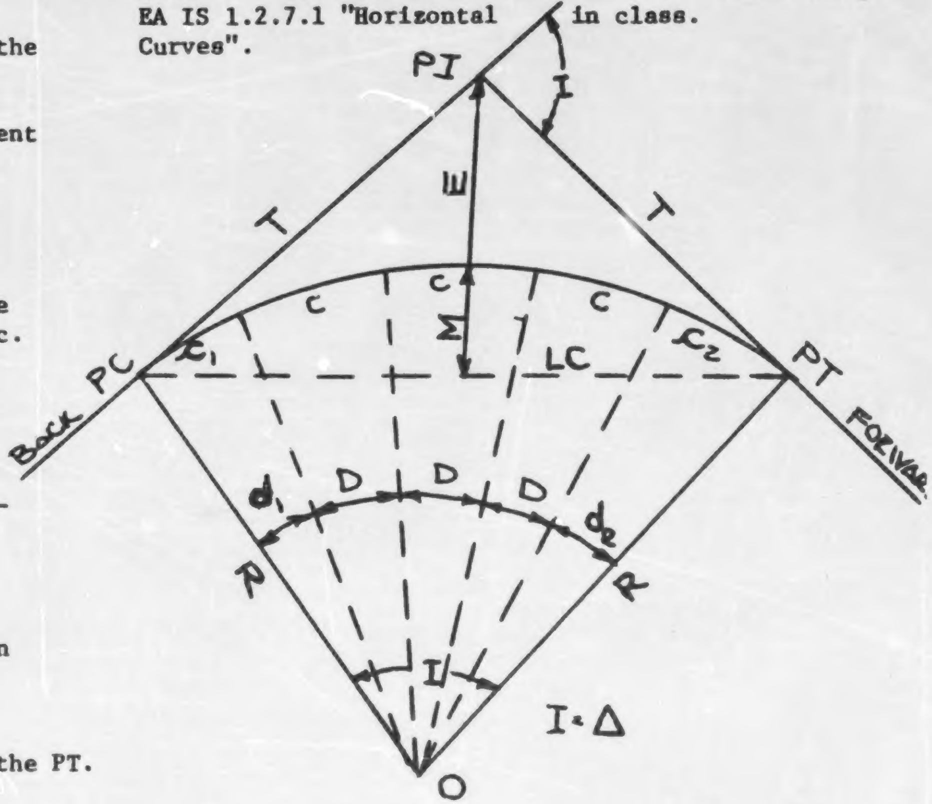
(1) The deflection angle between tangents at the point of intersection.

f. L. - Length of curve from PC to the PT.

g. T. - Length of tangent from the PI to either the PC or PT.

II.A.1. Pass out SCBT 410.2 EA IS 1.2.7.1 "Horizontal Curves".

II.A.1. Follow along in class.



OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

h. LC. - Length of long chord joining the PC with the PT.

i. R. - Radius of the curve.

j. D. - Degree of curve.

k. M. - Middle ordinate.

  (1) The distance from the mid-point of the arc to the mid-point of the LC.

l. E. - External Distance .

  (1) The distance from the PI to the mid-point of the arc.

m. C. - Length of full chord.

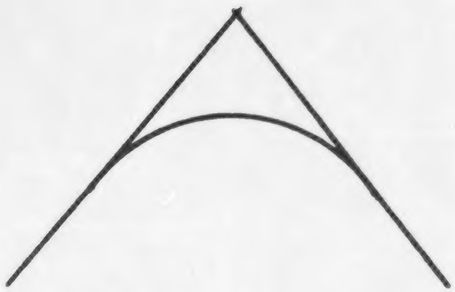
n. C_s - Length of sub chord.

O. d. - Central angle subtended by sub chord.

B. Types of Horizontal Curves.

1. Simple curve.

  (a) Arc of a circle which connects two points on two tangents equidistant from their points of intersection.



2. Reverse Curve.

   a. A continuous curve composed of two circular arcs with their centers on opposite sides of tangents common to both arcs at their meeting point.

3. Compound Curve.

   a. A continuous curve consisting of two circular arcs with centers on the same side of the tangent which is common to both arcs at their meeting points.

C. Degree of Curvature.

1. Arc Definition.

   a. The degree of curve is the angle formed by two radii drawn from the center of the circle to the ends of an arc 100 feet long.

   b. In this definition the degree of curve and radius are inversely proportional.

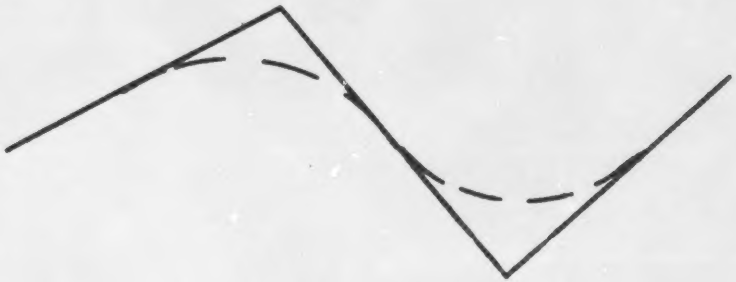
      (1) Example.

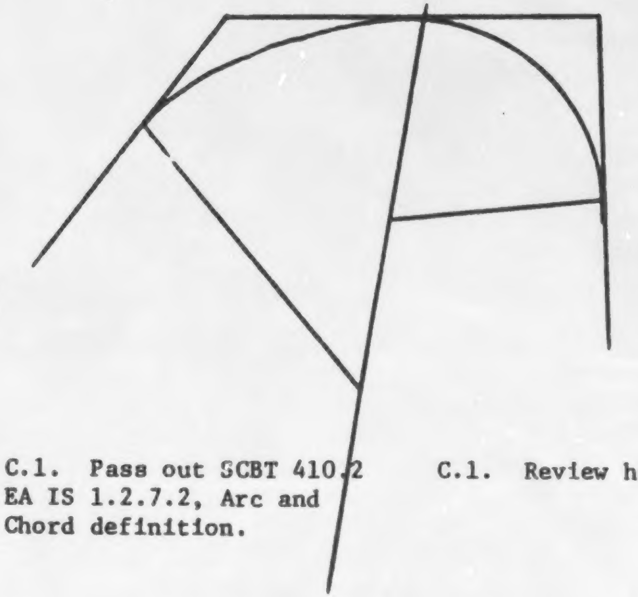
$D = 360^{o} :: \text{Arc} \quad \text{Circumference}$

Substituting $D = 1^{o}$

$1^{o} : 360^{o} :: 100 : 2R$

$1^{o} \; ; \; 360^{o} :: 100 : 6283185308 = 5129.58$ feet.





C.1. Pass out SCBT 410.2 EA IS 1.2.7.2, Arc and Chord definition.

C.1. Review handout.

5° degree curve, the radius = 1145-92 feet.

c. As degree of curve increase the radius increases.

d. It should be noted that for a given intersecting angle or central angle, all of the parts of the curve are inversely proportional to the degree of curve.

e. Used primarily for highways.

2. Chord Definition.

a. The degree of curve is the angle formed by two radii drawn from the center of a chord 100 feet long.

b. Radius formula

$50/\sin \ 1/2 \ D = R.$

(1) Example.

Assume $D = 1^{\circ}$

$50/\sin 0^{\circ} \ 30' = 50/0.0087265355 = 5729.65$ feet.

$D = 5^{\circ}$ the radius = 1145.28 feet.

c. The larger the degree of the curve, the sharper the curve and the shorter the radius.

d. Radius and degree or curve are not inversely proportional.

e. Used primarily for railroads but can be used for roads.

3. Chords.

   a. Used to layout curves due to the fact that it is impractical to stake the curve by locating the center of the circle and swing the arc with a tape.

   b. Ends of chords staked as they lie on the circumference of the curve.

   c. Length of chords vary with the degree of curve.

   d. To reduce the discrepancy between the arc distance and chord distance, the following chord lengths are commonly used:

      0° to 3° degree of curve - - 100 feet
      over 3° to 8° degree of curve - - 50 feet.
      over 8° to 16° degree of curve - - 24 feet
      over 16° degree of curve - - 10 feet.

      (1) Maximum distance in which the discrepancy between arc length and chord length will fall within the allowable error for taping which is .02' per 100 feet.

      (2) Longer or shorter chords can be used if conditions dictate.

4. Deflection Angles.

   a. The deflection angles are the angles between a tangent and the ends of chords, from the PC.

157

b. Used to locate chord direction.

c. The total of the deflection angles always equal to one half of I.

(1) Check.

D. Design of a Curve.

1. To solve a simple curve three elements must be known:

a. The PI

b. The I angle.

c. Normally the third part will be the degree of curve.

(1) Given in project specifications.

(2) Or computed based upon elements which have been limited by the terrain.

d. The PI and I angles are normally determined on the preliminary traverse for the road.

(1) May be determined by triangulation when the PI is inaccessible.

e. Example:

(1) Assume the following is known:

(a) PI = 18+00

(b) I = 75°

(c) D = 15°

(2) Solve the curve by both the arc and chord definition:

(a) $R = \frac{5729.58}{D}$ (Arc definition)

$R = \frac{5729.58}{15}$

R = 381.972

(b) T = R Tan 1/2 I.

T = R Tan 37° 30'

T = 381.972 .0.7673270

T = 293.10

(c) PC = PI - T

PC = 1800 - 293.10

PC = 15 = 06.90'

(d) $L = 100.\frac{I}{D}$ (Exact for arc deflection)

$L = 100 \frac{75}{15}$

L = 500'

(e) $PT = PC = L$

$PT = 15 = 06.90' = 500.00$

$PT = 20 = 06.90'$

(f) $E = T \tan 1/4\, I.$

$E = 293.10 - .3394543$

$E = 99.49$

(g) $M = R - (R \cos 1/2\, I)$

$M = 381.97 - (381.97 - 0.79335)$

$M = 78.93$

(h) $L_c = 2R \sin 1/2\, I.$

$L_c = 2 \times 381.97 \times 0.60876$

$L_c = 465.06$

(i) $R = \frac{50}{\sin 1/2 D}$ Chord definition

$R = \frac{50}{0.13053}$

$R = 383.05$

(j) $T = R \tan 1/2\, I$

$T = 383.05 \times 0.76733$

$T = 293.93$

(k) $PC = PI - T$

$PC = 18 = 00 - 2 + 93.93$

$PC = 15 = 06.07$

(1) $L = 100 \frac{I}{D}$

$L = 100 \times \frac{75}{15}$

$L = 500$

(m) $PT = PC = L$

$PT = 15 = 06.07 = 500$

$PT = 20 = 06.07$

(n) $E = R \text{ EXSEC } 1/2\, I$

$E = 383.05 \times 0.26047$

$E = 99.77$

(o) $M = R \text{ Vers } 1/2\, I$

$M = 383.05 \times 0.20665$

$M = 79.16$

(p) $LC = 22 \text{ Sin } 1/2\, I$

$LC = 2 \times 383.05 \times 0.60876$

$LC = 466.37$

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

(q) d1 & d2 Arc definition

d1 = 0.3CD

d1 = 0.3 x 43.10 x 15 = 3° 13.95'

d2 = 0.3 x 6.90 x 15 = 0° 31.05'

(r) d1 & d2 Chord definition

d1 = 0.3CD

d1 = 0.3 x 43.93 x 15 = 3° 17.685'

d2 = 0.3 x 6.07 x 15 = 0° 27.315

(s) Deflection angle

d50 = 0.3 x CD

d50 = 0.3 x 50 x 15

d50 = 3° 45'

(t) Check

Arc Def.	Chord Def.
d1 3° + 13.95	d1 3° 17.685'
d2 0° + 31.05'	d2 0° 27.315'
d50 = 33° 45.00	d50 33° 45.00'
37° 30.00	37° 30.00'

f. How to use table of function for a 1° curve and table of correction to be added for values of T; E in (arc definition) computations, when chord definition will be used.	D.2.f Pass out SCBT 410.2 EA IS 1.2.7.5.	D.2.f Follow along in handout.

(1) Example:

Given D^2 30°; A = 45°

(a) By formula:

1. Arc Def. $R = \frac{5729.58}{D}$ $\frac{5729.58}{30^\circ}$ = 190.99'

2. Chord Def. $= \frac{50}{\sin \frac{D}{2}}$ $\frac{50}{\sin 15^\circ}$ 193.19'

From 1. $T = R \tan \frac{I}{2} = 190.99 \tan 22^\circ - 30' = 79.11'$

From 2. $T = R \tan \frac{I}{2} = 193.19 \tan 22^\circ - 30' = 80.02$

(b) By use of table Under 45° on T we find a value T = 2373.4 and dividing this value by D, thus:
$T = \frac{2373.4}{30} = 79.11'$ (same value from the arc def. above)

.: Values from formula and by the use of tables are equal as both applied with arc definition.

(c) Used correction table to apply the chord definitions thus:

I=A	D 30°
40	0.80
45	0.91
50	1.02

.: T = 79.11 + 0.91

T = 80 - 02'

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
E. Field notes (setting up) 1. Example.	II.E.1. Pass out SCBT 410.2 EA IS 1.2.7.4 Note format.	II.E.1 Follow along with handout.
F. Field procedures for layout of circular curves.		
1. To layout a circular curve (arc definition) the usual procedure is as follows:		
a. With the instrument at the PI, the instrumentman sights on the preceeding PI or at a distant station and keeps the chainman on line while the tangent distance is measured to locate the PC. Sfter the PC has been staked, the instrument is then trained on the forward PI and the PT is located.		
b. The instrument is then set up at the PC and the angle from the PI to the PT is measured. This angle should be equal to one-half the I angle; if not, either the PC or PT has been located in the wrong position.		
c. With the first deflection angle set on the plates, the instrumentman keeps the chainman on line as the first subchord distance is measured from the PC.		
d. Without touching the lower motion, the second deflection angle is set on the plates, the chainmen measure the chord from the previous station while the instrumentman keeps the head chainman on line.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
e. The succeeding stations are staked in the same manner. If the work is done correctly, the last deflection angle will point on the PT and the distance will be the subchord length from the last station prior to the PI.		
III. Application.		
A. Discussion	III.A. Questions.	III.A Answers
	1. Define PI.	1. Point of intersection of the two tangents.
	2. Define PC.	2. Point of change from curve to tangent.
	3. Define a reverse curve.	3. A continuous curve composed of two circular arcs with their centers on opposite sides of tangents common to both arcs at their meeting point.
	4. The arc definition is used primarily for?	4. Highways.
	5. The chord length recommended for a over 3° to 8° degree of curve is?	5. 50 feet.
	6. What three elements must be known to solve a simple curve?	6. PI, I and normally the degree of curve.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
B. Practical Performance.	III.B. Read and explain criterion test. Have students design required curve and layout in the field.	III.B. Design and layout curve.
1. Have each student design curve for road intersection developed in previous topics.		
2. Have each student set up field notes.		
3. Check all office work.		
4. Have each survey party layout curve in filed.		
IV. Summary.		
A. Horizontal Curves.		
B. Types of horizontal curves.		
C. Degree of curvatures.		
D. Design of a curve.		
F. Setting up notes.		
F. Field procedures for layout of circular curves.		
V. Test.		
A. None		

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INFORMATION SHEET

TITLE: Horizontal Curves.

TERMINOLOGY:

1. PC The POINT OF CURVATURE. The point of curvature, indicated by the initials PC, is the point where the circular curve begins. The back tangent to the curve at this point.

2. POC POINT ON CURVE. This is any point along the curve and is indicated by the initials POC.

3. PT The POINT OF TENGENCY. The point of tangency is the end of the curve. Is is indicated by the initials PT. The forward tangent is the tangent to curve at this point.

4. L The LENGTH OF CURVE. The length of curve is the distance from the PC to the PT measured along the curve.

5. T The TANGENT DISTANCE. The tangent distance is the distance along the tangents from the PI to the PC or PT. These distances are equal on a simple curve.

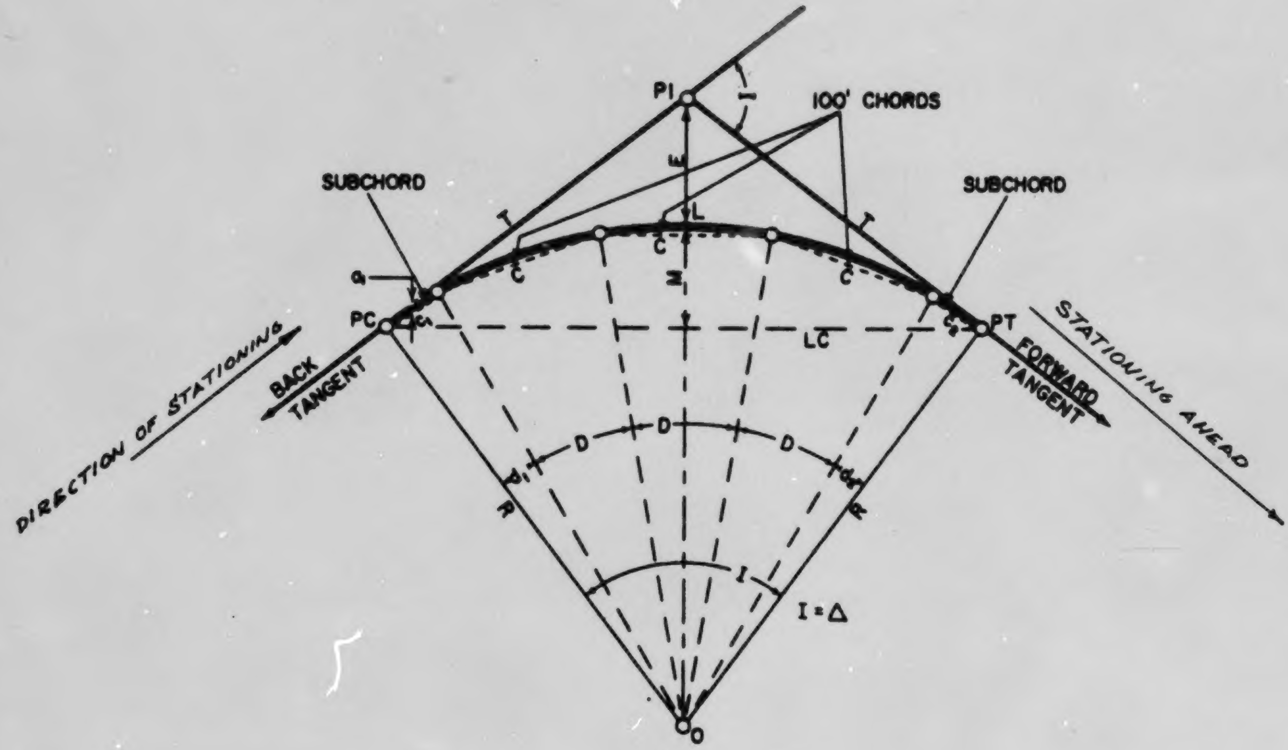
6. The CENTRAL ANGLE. The central angle is the angle formed by two radii drawn from the center of the circle (0) to the PC and PI. The central angle is equal in value to the I angle. Some authorities call both the intersecting angle and central angle either I or

7. LC LONG CHORD. The long chord is the chord from the PC to the PT.

8. E EXTERNAL DISTANCE. The external distance is the distance from the PI to the midpoint of the curve. The external distance bisects the interior angle at the PI.

9. M MIDDLE ORDINATE. The middle ordinate is the distance from the midpoint of the curve to the midpoint of the long chord.

10. D DEGREE OF CURVE. The degree of curve defines the "sharpness" or "flatness" of the curve.



FORMULAS

$R = (\text{chord def.}) = \dfrac{50}{\text{Sin } 1/2\ D}$

$R = (\text{arc def.}) = \dfrac{100 \times 360}{2 \quad D} = \dfrac{5729.58}{D}$

$T = R \text{ Tan } \dfrac{I}{2}$

$T = \dfrac{E}{\text{Tan } \dfrac{I}{4}}$

$E = T \text{ Tan } \dfrac{I}{4}$

$LC = 2R \text{ Sin } \dfrac{I}{2}\ ;\ = 2T \text{ CDS } \dfrac{I}{2}$

$L \text{ (in feet)} = 100 \dfrac{I}{D}$

$L \text{ (in station)} = \dfrac{I}{D}$

$d, = \dfrac{C_1}{100} (D)$

$d_2 = \dfrac{C_2}{100} (D)$

$a_1 = \dfrac{C_1}{C} \times \dfrac{D}{2}$

$$E = \frac{R}{\cos \frac{I}{2}} - R$$

$$E = R \text{ EXSEC } \frac{I}{2}$$; where:

$$\text{EXCES } \frac{I}{2} = \text{SEC } \frac{I}{2} - 1$$

$$M = R - (R \cos \frac{I}{2})$$

$$M = R \text{ VERS. } \frac{I}{2}$$; where

$$\text{VERS } \frac{I}{2} = 1 - \cos \frac{I}{2}$$

.: $a_1 = {}^{*}\, 0.3\ C_1\ D$

$$C_1 = 2 \left(\frac{360}{d_1}\right) \left(\frac{A_1}{2}\right) \text{Sin } \frac{d_1}{2}$$

* Constant value

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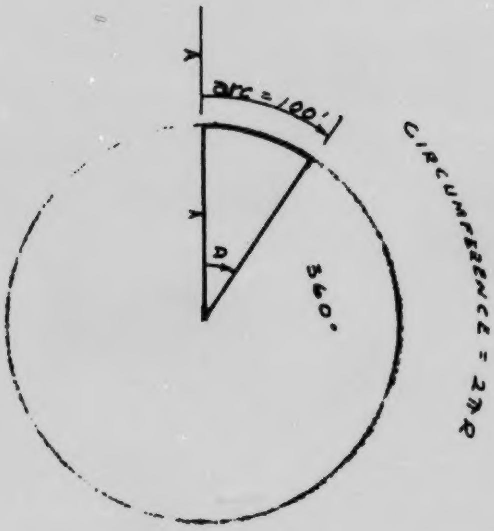
INFORMATION SHEET

TITLE: ARC AND CHORD DEFINITION



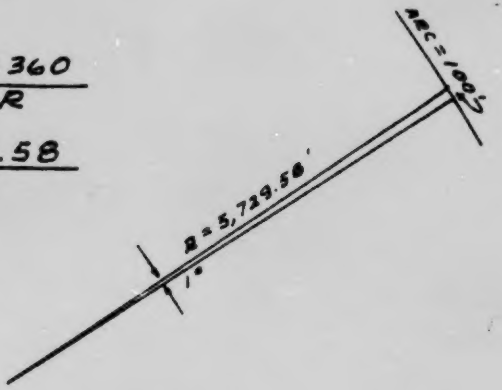
A. ARC DEFINITION:

$$\frac{D}{360} = \frac{100}{2\pi R}$$

SOLVING FOR R:

$$R = \frac{100 \times 360}{2\pi R}$$

$$R = \frac{5729.58}{D}$$

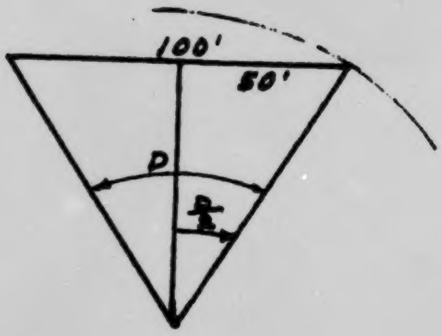


FOR 1° CURVE, D = 1

$\therefore$

$$R = \frac{5729.58}{1}$$

$$R = 5729.58$$



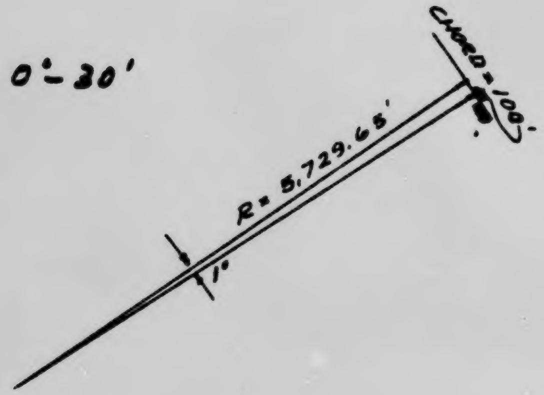
B. CHORD DEFINITION:

$$\sin\frac{D}{2} = \frac{50}{R}$$

SOLVING FOR R

$$R = \frac{50}{\sin\frac{D}{2}}$$

FOR A 1°; $\frac{D}{2} = 0^\circ\text{-}30'$



∴

$$R = \frac{50}{\sin .\, 0^\circ\text{-}30'}$$

$$R = 5729.65'$$

CONCLUSION:

Therefore, there is a .07 difference in R values between arc definition and chord definition for 1° curve. For some calculation not requiring great precision the rounded-off value of 5730' is used for R of a 1° curve.

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INFORMATION SHEET

TITLE: Horizontal Curve Formulas.

OBJECTIVE: To provide a ready reference.

1. The following formulas are used in the computation of a simple curve. All the formulas apply to both the arc and chord definitions except those noted.

A. $R = \frac{5729.58}{D}$ (Arc definition)

B. $R = \frac{50}{\text{SIN } 1/2\ D}$ (Chord definition)

C. $T = R \text{ Tan } 1/2\ I$.

D. $L = 100 \frac{I}{D}$ (Exact for Arc depletion)

E. $PC = PI - T$.

F. $PT = PC + 1$.

G. $E = R \text{ Exsec } 1/2\ I$.

H. $E = T \text{ Tan } 1/4\ I$.

I. $E = \frac{R}{\text{COS } 1/2\ I} - R$.

J. $M = R - (R \text{ COS } 1/2\ I)$.

K. $M = R \text{ Vers } 1/2\ I$.

L. 0.3 CD --- Exact for the arc definition. Approximate for chord definition. This formula gives an answer in minutes.

M. $L_c = 2 R \text{ SIN } 1/2\ I$.

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INFORMATION SHEET

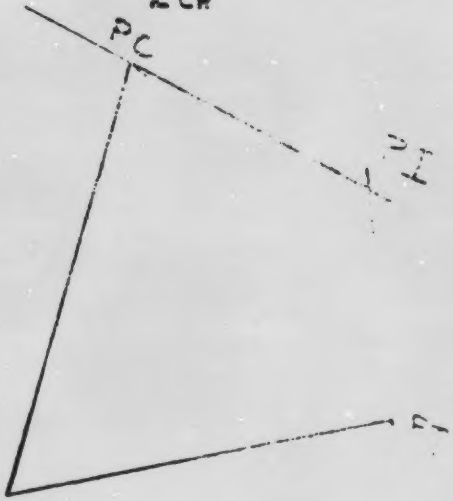
TITLE: SAMPLE NOTE FORMAT (HORIZONTAL CURVE)

STA	CHORD	DEF ANGLE
15+06.90 PC	—	—
15+50	C_1 43.10	d_1 3°14'
16+00	50	6°59'
16+50	50	10°44'
17+00	50	14°29'
17+50	50	18°14'
18+00	50	21°59'
18+50	50	25°44'
19+00	50	29°29'
19+50	50	33°14'
20+00	50	36°59'
20+06.9 PT	C_2 06.90	d_2 37°30'

CLOUDY, COOL

DATE
⊼
[illegible]
H CH
R CH

PI = 18+00
Δ = 75°00'
D = 15°00'
T = 293.10'
L = 500'
R = 381.97'
d_1 = 3° 13.95'
d_2 = 0° 31.05'
d_{50} = 3° 45'



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INFORMATION SHEET

Curve computation and Correction tables.

OBJECTIVE: To explain how to use table of functions for a 1 degree curve and a table of correction to be added for values of T ; R in (Arc definition) computations, when chord definition will be used.

Example: Given $D = 30^{\circ}$; $I - 45^{\circ}$.

I BY FORMULA

a. ARC DEF. $R = \frac{5729.58}{D} = \frac{5729.58}{30^\circ} = 190.99'$

b. CHORD DEF. $R = \frac{50}{\sin \frac{D}{2}} = \frac{50}{\sin 15^\circ} = 193.19'$

FROM ⓐ $T = R \tan \frac{\Delta}{2} = 190.99 \tan 22^\circ\text{-}30' = 79.11'$

ⓑ $T = R \tan \frac{\Delta}{2} = 193.19 \tan 22^\circ\text{-}30' = 80.02'$

IIa. BY THE USE OF TABLE UNDER 45° ON T. WE FIND A VALUE T = 2373.4 AND DIVIDING THIS VALUE BY D, THUS:

$T = \frac{2373.4}{30} = 79.11'$ (SAME VALUE FROM THE ARC DEF. ABOVE.)

∴ VALUES FROM FORMULA AND BY THE USE OF TABLE ARE EQUAL AS BOTH APPLIED WITH ARC DEFINITION

b. USED CORRECTION TABLE TO APPLY THE CHORD DEFINITION

THUS:

Δ	D 30°
40	0.80
45	0.91
50	1.02

∴

$T = 79.11 + 0.91$

$T = 80.02'$

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Classification: Unclassified.

Topic: Vertical Curves.

Average Time: 2 Periods (Class) 5 Periods (Pract)

Instructional Materials:

A. Texts:

1. EngineeringAid 1 & C, NAVEDTRA 10635-B.

B. References:

1. Surveying, Legault, McMaster & Marlette.

2. Surveying Theory & Practice, 5th Edition, Davis, Foote & Kelly.

C. Tools, Equipment and Materials:

1. See Annex III.

D. Training Aids & Devices:

1. Films: None

2. Transparencies: None.

3. Charts: None.

4. Locally Prepared Materials.

a. Information Sheets.

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acte as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual top objectives.

Enabling Objectives: Upon completion of this topic each student will be able to design a vertical curve and set up field notes for use by a survey party; consisting of four (4) students who will perform all the duties of an instrumentman, chainman and notekeeper; in laying out the vertical curve on the road centerline established by top 1.2.5 "Road Surveying" and 1.2.6 "Road Stakes". Design and layout procedures and computations will be as specified in this instructor's guide and Engineering Aid 3 & 2, NAVPERS 10634-B. Students will be 100 percent correct for all computations and procedures for designing the vertical curve and in setting up the field notes; al field work performed by the survey party will be to 3rd order precision.

Criterion Test: Each student will design a vertical curve and set up field notes for use by a survey party consisting of four (4) students, perform all the duties of instrumentman, chainman and notekeeper in laying out the curve on the road centerline, established by topics 1.2.5 "Road Surveying" and 1.2.6 "Road Stakes". Design and layout procedures and computations shall be as specified in this instructor's guide and Engineering Aid 3 & 2 Navpers 10634-B. Students will be 100 percent correct.

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(1) SCBT 410.2 EA IS 1.2.8.1, Sample Note Format (Vertical Curve).

b. Problem Sheets.

(1) SCBT 410.2 EA PS 1.2.8.1, Vertical Curves.

E. Training Aids Equipment:

1. None.

for all computations and procedures for designing the curve and setting up of the field notes, all field work performed by the surveying party will be to 3rd order precision.

Homework: Read

Engineering Aid 1 & C, NAVEDTRA 10635-B, Chapter 8, pp 230-240

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Vertical Curves.		
B. Establish readiness.	I.B. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advnaced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to compute and layout in the field a vertical curve.	I.D.1. State information and materials necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

II. Presentation.

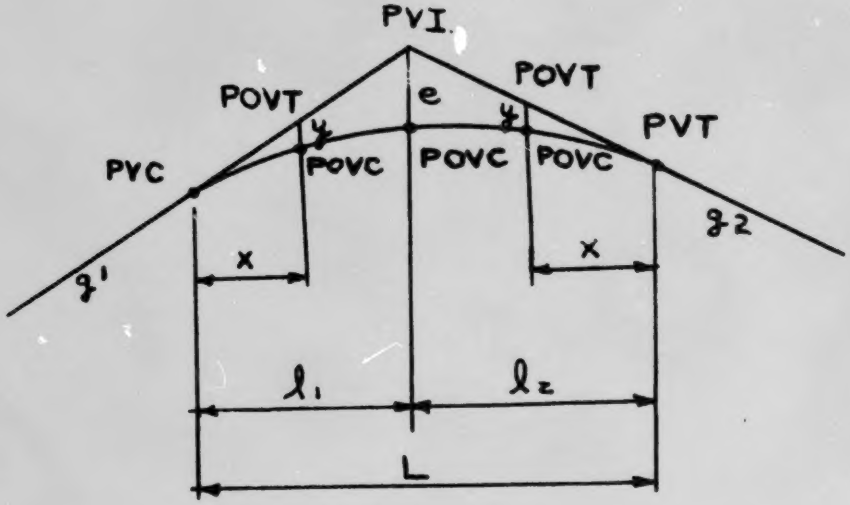
A. Vertical Curves.

1. Definition - When two grade lines intersect there is a vertical change of direction. The intersection is rounded by inserting a vertical parabolic curve. The curve makes the direction change gradual from one grade to the next.

B. Vertical Curve Terminology.

1. PVC Point on Vertical Curve.
2. PVI Point of Vertical Intersection.
3. PVT Point of Vertical Tangency.
4. g_1 Entering Grade (%)
   - a. +3.25 grade raises 3.25 feet per 100 feet.
   - b. -3.25 grade drops 3.25 feet per 100 feet.
5. G Gradient, Algebraic difference of g_1 and g_2 (A)
6. g_2 Leaving grade, figured same as g_1.
7. L -Horizontal length in 100 foot stations from PVC to PVT.
8. l_1 -Horizontal length from PVC to PVI measured in feet.

   l_2 -Horizontal length from PVI to PVT measured in feet.



9. e Vertical (External) distance from the PVI to the curve measured in feet.

$$e = \frac{LG}{8}$$

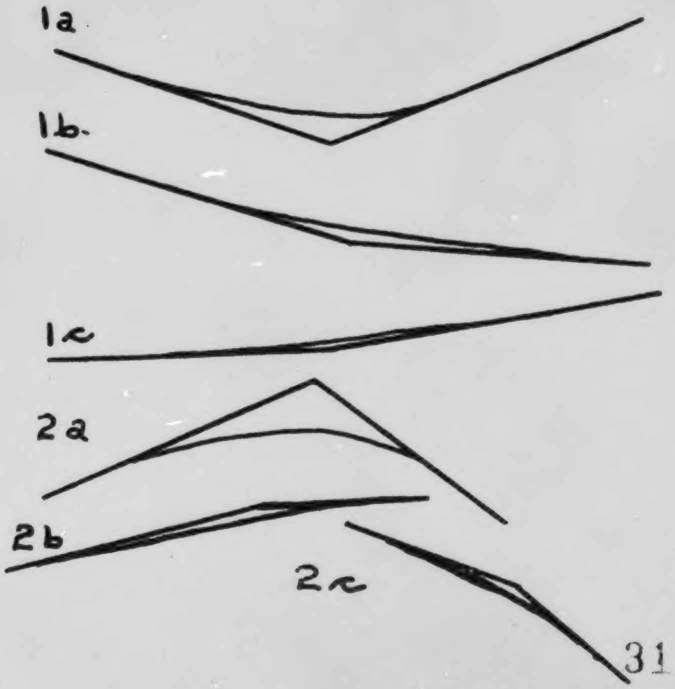
10. x Horizontal distance from PVC or PVT to point on curve (station) distance never greater than 1/2 L.

11. y Vertical offset, difference in elevation between tangent and point on vertical at x distance.

C. Types.

1. Sag curve.
    - a. A descending grade followed by ascending grade.
    - b. A descending grade followed by one descending less sharply.
    - c. An ascending grade followed by one ascending more sharply.

2. Summit Curve.
    - a. An ascending grade followed by a descending grade.
    - b. A descending grade followed by one descending more.
    - c. Ascending grade followed by one asecnding more sharply.



OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

3. Symmetrical Vertical Curves.

   a. Distance from PVC to PVI equals distance from PVT to PVI or $l_1 = l_2$

4. Unsymmetrical Vertical Curves.

   a. Distance from PVC to PVI does not equal distance from PVT to PVI or $l_1 = l_2$.

   b. Formulas change for unsymmetrical vertical curves.

D. Formulas for Vertical Curves.

NOTE: These formulas apply only to long radius curves.

1. Formulas.

   a. $A = g_2 - g_1$ (Algebracialy)

   b. $L = \frac{g_2 - g_1}{r} = \frac{A}{r}$ (The curve length in station)

   c. $r = \frac{A}{L}$ (L being extended to the L nearest whole stations)

   d. $M = \frac{AL}{8}$

   e. $y = M \frac{X}{\frac{L}{2}}$

   y - is the offset distance

   x is the distance to the curve ends and may be expressed as a proportion of the distance from the PVC or the PVT.

OUTLINE OF INSTRUCTION

f. $xe = \frac{g}{r} = \frac{gL}{A}$

When the grades are equal, these points are directly above or below the PVI; but with unequal grades, these points will lie between PVI and the part of the curve which is on the flatter gradient. The x value to these points (xe) is determined by this formula in which g is the grade of the lesser slope.

2. Computations.

a. Set Up Note Sheet.

(1) Column 1 - Stations

(2) Column 2 - Tangent Elevation.

(3) Column 3 - Ration of x/l_2

(4) Column 4 - Ratio of (x/l)

(5) Column 5 - Vertical offset $(x/e)^2$ (e)

(6) Column 6 - grade elevation on the curve.

(7) Column 7 - First difference.

(8) Column 8 - Second defference.

b. Starting with the PVC station figure tangent elevations to the PVI rather than from the PVI to the PVT.

INSTRUCTOR ACTIVITY

II.2.a. Pass out SCBT 410.2 EA IS 1.2.8.1 Note Format

STUDENT ACTIVITY

II.2.a. Fill in as class example is worked

(1) g_1 and g_2 are given as percents. The percents equals the slope per 100 feet.

(a) Given problem information

g_1 = +9%
g_2 - 7%
L = 400.00', or 4 Stations.
PVI = 30 + 00
PVI Elevation = 239.12 ft.

(b) Solution

$l_1 = l_2$ = Symmetrical curve

l_1 = 200 feet = 2 x 9 = 18 feet.

l_2 = 200 ft. = 2 x 7 = 14 feet.

Elevation PVC = 239.12 - 18
PVC = 221.12 feet
Elevation PVT = 239.12 - 14
PVT = 225 - 12 feet

(2) By adding the gradients or subtracting them at each station the tangent elevation at the next station can be obtained.

(a) Determine stations - 50 feet interval.

30+00 - 2+00 = 28+00 PVC
30=00 + 2+00 = 32+00 PVT

(b) g_1 100 feet = 9 feet rise
50 feet = $\frac{9}{2}$ = 4.50 feet

g_2 100 feet = 7 foot fall.
50 feet = $\frac{7}{2}$ = 3.50 feet

b. Solution

PVC 28+00 = 221.12 feet
28+50 = 221.12+4.50=225.62 feet
29+00 = 225.62+4.50=230.12 feet
29+50 = 230.12+4.50=234.62 feet

PVI 30+00 = 234.62+4.50=239.12 feet
30+50 = 239.12-3.50=235.62 feet
31+00 = 235.62-3.50=232.12 feet
31+50 = 232.12-3.50=228.62 feet

PVT 32+00 = 228.62-3.50=225.12 feet

(3) Calculate (e), The middle vertical offset at the PVI.

(a) First find G.

$G = g_2 - g_1 -$

$G = -7 - (+9)$

$G = -16\%$

c. Solution

$$e = \frac{LG}{8}$$

$$e = \frac{(4)\ (-16)}{8} = -8.00 \text{ ft.}$$

The negative sign indicates "e" is to be subtracted from the PVI.

(4) Compute the Vertical Offset at each 50 - foot station, using the formula $y = \left(\frac{x}{1}\right) 2.\ e$

(a) Compute $\frac{x}{1}$, then square it, Then multiply by "e"

Station 28+00 = $\frac{50}{200}$ = 1/4 = $(1/4)^2$ = [illegible]/16

(b) Solution

Vertical offset at station 28=50= (1/16) (-8) = - 0.50

1. Repeat for all stations.

(5) Compute the grade elevation at each 50 foot stations.

(a) Offset is negative, therefore subtract.

(b) Solution - example

28+50 = -0.50
28+50 elevation = 225.62 - 0.50
Curve elevation = 225.12 feet.

(c) Complete curve elevation computations.

(6) Find the turning point on the vertical curve.

(a) Highest or lowest point.

(b) Formula

$$X_t = \frac{gL}{G}$$

X_t = Distance of turning point from PVC or PVT.

g = Lesserslope (ignoring signs)

L = length of curve in stations.

G = Algebraic difference of slopes.

(c) Solution (Turning point).

$$X_t = \frac{gL}{G} = \frac{7\ (4)}{16} = 1.75$$

Turning point in 175 feet from PVT or station 30+25

(d) Solution (Turning point offset)

$$Y_t = \left(\frac{X_t}{1}\right)^2 e = \frac{(1.75)^2}{2}\ 8 = 6.12$$

POVT at 30+25 = 237.37 - 6.12

POVT at 30+25 = 231.25 feet.

(7) Check work.

(a) Subtract each grade elevation from the preceeding or following grade elevation to obtain the first difference.

(b) Subtract each first difference from the preceeding or following first difference to obtain the second difference, the second difference should equal or be reasonably close to equal, this serves as a check on computations.

E. Field Stakeout of Vertical Curves.

1. Basically of marking finished elevations in the field to guide construction personnel.

2. Any field change can be computed in field, using data presented in this instructor guide.

III. Application

A. Discussion

III.A. Questions	III.A. Answers
1. Name two types of vertical curves.	1. Summit, sag.
2. Define L.	2. Horizontal length in 100 foot stations from PVC to PVT.
3. Define POVT	3. Point on vertical tangent.
4. Define G.	4. Gradient, Algebraic difference of g_1 and g_2

OUTLINE OF INSTRUCTION

B. Practical Performance

1. Perform requirements of SCBT 410.2 EA PS 1.2.8.1

2. Design a vertical curve when given Station at the PVI is 225 feet from both the PVC and PVT.

l_1 = 225 feet

l_2 = 225 feet

g_1 = 0.0%

g_2 = 5.3%

IV Summary

A. Vertical Curves.

B. Vertical Curve Terminology

C. Types.

D. Formulas for Vertical Curves.

E. Field Stakeout of Vertical Curves

V. Test.

A. None

INSTRUCTOR ACTIVITY

III.B. Pass out SCBT 410.2 EA PS 1.2.8.1 and have students perform.

III.B.2 Read and explain criterion test. Have students design set up field notes and layout curve in the field.

STUDENT ACTIVITY

III.B. Perform requirements of SCBT 410.2 EA PS 1.2.8.1

III.B.2 Perform requirements of criterion test.

SCBT 410.2 EA IS 1.2.8.1

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INFORMATION SHEET

TITLE: Sample Note Format (Vertical Curve)

Station	Elevation or Tangent	$\frac{X}{e}$	$\left(\frac{X}{e}\right)^2$	Vertical Offset	Grade Elevation on Curve	First Elevation	Second Elevation

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PROBLEM SHEET

TITLE: Vertical Curves

OBJECTIVES: This problem sheet has been prepared to give you needed practice in the design of vertical curves. Successful completion of this problem sheet is a good indication that you are capable of designing vertical curves.

PROCEDURES:

1. Use the problems worked in class as a guide. All needed information for the solution will be given. Show all work. If you have any questions raise your hand for instructor assistance.

2. For the following vertical curves calculate A, M, L, r (if not given), curve elevations. Show work in tabular form, including check. Show sketch.

(a) Given: PVI = Sta. 23+00, elev = +150.00,
r = change in grade/station = 1%/station,
$g_1 = +1\%$, $g_2 = +7\%$
Compute curve elevations at 100' stations.

Sta.	Tan. El.	Computation	Offset	Curve El.	1st Diff	2nd Diff

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(b) Given: PVI = Sta. 73+00, elev = +100.00
r must be no greater than 1.25%/100' station,
$g_1 = +5\%$, $g_2 = -4\%$,
Use L in even whole stations (2, 4, 6, 8, 10, etc.) such that r is approximately (but not greater than) 1.25%/ station.
Compute curve elevations at 50' intervals.

Sta.	Tan. Elev	Computation	Offset	Curve El.	1st Diff	2nd Diff

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Grade and Earthwork Computations.

Average Time: 2 Periods (Class) 5 Periods (Pract)

Instructional Materials:

A. Texts:

1. Engineering Aid 3 & 2,NAVPERS 10634-B

B. Reference:

1. Surveying, Legault, McMaster, Marlette.

2. Surveying, Davis and Foote, 4th Edition

C. Tools, Equipment and Materials:

1. See Annex III.

D. Training Aids & Devices:

1. Film: None

2. Transparencies: None

3. Charts: None

4. Locally Prepared Materials:

a. Information Sheets.

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. Each student will have acte as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic the student will have fixed grade as specified by the instructor, computed earthwork volumes using both the "AVERAGE END AREA" and "PRISMOIDAL FORMULA" methods and draw the mass diagram. The student will use the rough profiles and cross sections from Topic 1.2.4 "Road Surveying", instructor guidance and Engineering Aid 3 & 2, NAVPERS 10634-B in meeting the requirements of this topic Each student will perform his own calculations. All math computations will be 100% percent, all earthwork volumes as individually calculated, will be within 5% percent as a party for accuracy.

Criterion Test: Each student will fix grade, as specified by the instructor for the road established in Topic 1.2.4, draw finish profiles and crass sections and do the related earthwork computations and draw the mass diagram. Each student will do his/her own calculations, use both the "AVERAGE END AREA" and the "PRISMOIDAL FORMULA" methods. All math computations will be 100% percent accurate, all earthwork volumes as individually calculated, will be within 5% percent as a party for accuracy.

(1) SCBT 410.2 EA IS 1.2.9.1 Volume Determination

(2) SCBT 410.2 EA IS 1.2.9.2 Example Converted Soil Volumes

(3) SCBT 410.2 EA IS 1.2.9.3 Mass Diagram

E. Training Aids Equipment:

1. None

Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B pp 146-151

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.	I.A. Introduce self and topic.	
A. Establish contact.		
1. Name:		
2. Topic: Grade and Earthwork Computations.		
B. Establish readiness.	I.B. Motivate student.	
1. Purpose		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to fix grade, draw finished profile and cross sections and do related earthwork calculations.	I.D. 1. State information and materials necessary to guide student.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

II. Presentation

a. Grade

1. Has two meanings in construction.

   a. Grade or rate of grade, meaning the rise and fall in feet per 100 feet.

   b. The grade of a building foundation floor, etc. meaning the elevation of a point with respect to the plane of reference being used.

   c. In road surveying "a" is the grade term of vital interest.

2. Grade is used to control or guide the construction effort.

3. Fixing the grade.

   a. Profiles furnish the basis for study of economic grade elevation.

   b. Other considerations

      (1) Traffic requirements.

      (2) Fixed or narrow limits such as terminal points, streams, railroad crossings, etc.

      (3) Usable space.

   c. Field and office work is simplified if rates of grade are expressed in and exact decimal, as 2.5 percent or 0.65 percent.

d. Once all considerations are known grade lines are drawn on rough profiles and by trial and error a satisfactory solution is found which satisfies all restrictions.

B. Earthwork Computations.

1. Purpose

a. Planning and Estimating

b. Quanity of earth to be moved.

c. Equipment to be used determined.

d. Time

e. Depending upon quality of fill balancing the cut and fill can be accomplished.

2. Cross sections.

a. End areas views are plotted on graph paper and taken as a perpendicular section along the center line.

(1) Full stations.

(2) Culverts

(3) Bridges

(4) PC's

(5) PT's

(6) Etc.

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OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
b. Most common scale is 1" = 5' vertical and 1" = 10' horizontal.		
3. End area computations.		
a. Geometric method.		
(1) Divide each cross-section into triangles, rectangles and trapezoids whose areas are figured separately and then totaled to determine a square footage.		
(2) No set rule for subdividing the end areas.	II.B.3.a.(2) Draw cross-section and demonstrate on C/B.	
(a) Choose those subdivisions which produce the most direct and accurate results.		
b. Mechanical Method-employs the compensating polar planimeter.		
(1) Planimeter has a tracing arm that is attached to the revolving drum.		
(a) Drum graduated into a hundred parts and can be read to thousandths by the use of the vernier.		
(2) Most planimeters are set to the 10 scale.		
(a) One complete drum revolution equals 10 square inches.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
(b) Each planimeter should be checked before use by tracing around a pre-determined area on cross section paper.		
(3) Procedures.	II.B.3.b.(3) Demonstrate procedures by having class perform each step.	II.B.3.b.(3) Perform each step as instructed.
(a) Take initial reading		
(b) Trace cross-section as smoothly as possible.		
(c) Take second reading and subtract it from the first reading.		
(d) Retrace cross-section.		
(e) Take third reading and subtract it from the second.		
1. If readings are not reasonably close retrace again.		
(f) Average the two closest readings and multiply by constant.		
1. Constant is horizontal scale multiplied by vertical scale.		
(g) The answer is the area of the cross-section in square feet.		

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

c. Stripper method - very good for small cuts or fill.

II.B.3.c. Demonstrate on C/B.

(1) Stripper is made by making each square along a cumulative length.

(2) Stripper is moved across the section in intervals of 3,5, or 10.

(3) The top reading on the stripper is placed on the bottom of each new interval until it has been moved across the entire section.

(4) The last reading is multiplied by the interval (3, 5 or 10) that was used to obtain the number of squares in the section.

(5) Multiply the number of squares by the area of one square to find the area of the cross-section in square feet.

C. Volume Determinations:

1. Average End Area Method.

a. This method assumes that the volume between two adjacent sections is the average of their two end areas multiplied by the distance between them.

$$V = \left(\frac{A_2 + A_1}{2}\right) L$$

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

b. This-formula is only correct when $A_1 = A_2$, but is sufficiently accurate for earthwork computations.

c. End areas are in square feet and the distance between stations is in feet so the formula is modified to give an answer in cubic yards.

$$V = \left(\frac{A_1 + A_2}{54}\right) L$$

2. Prismodial Formula.

a. Volume of a prismoid

$$V = \frac{L}{2}\left(A_1 + 4AM + A_2\right)$$

L = Distance between stations.

A_1 = End area one.

A_2 = End area two.

AM = Middle end area halfway between the end sections AM is determined by averaging corresponding linear dimensions of the two end sections.

b. Example problem:

II.C.2.b. Pass out SCBT 410.2 EA IS 1.2.9.1 Volume Determination.

II.C.2.b. Follow along on information sheet.

c. Prismodial formula is more accurate.

(1) Only justified when cross-sections are taken at short intervals, surface deviations are noted, or if successive sections vary widely.

d. A prismodial correction for volumes computed by the average end area may be done by using:

$Cv = .309 (H_o - H_1) \quad (D_o - D_1)$

Cv = Difference in volume, or correction for prismodial 100 feet ling in cubic yards.

H_o = Center height at one end section, in feet.

H_1 = Center height at the other end section in feet.

D_o = Distance between slope stakes at the end section where the center height is H_o, in feet.

D_1 = Distance between slope stakes at the other end section, in feet.

(1) Solving for the previous example:

$Cv = .309 (6.0 - 3.0) (44.0 - 35.0) = 8.3$ cubic yards.

D. Net Volumes by Using Earthwork Factors.

1. Shrinkage.

a. One cubic yard of soil in place will not equal one cubic yard after compaction.

352 b. Shrinkage is due to transportation loss and compaction.

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

c. Common shrinkage factor is 90%, but can be as low as 70%

2. Swell.

   a. Some soils swell during transportation or stockpiling.

   b. Swell may be from 10% to 40%.

II.D.2. Pass out SCBT 410.2 EA IS 1.2.9.2, Example Converted Soil Volumes.

E. Mass Diagram

1. The mass diagram is a graph or curve on which the algebraic sums of cuts and fills are plotted against linear distance.

   a. Cuts are indicated by a rise in the curve and are considered positive.

   b. Fills are indicated by a drop in the curve, and are considered negative.

   c. The yardage between any pair of stations can be determined by inspection.

      (1) Balances cuts and fills within the limits of economic haul.

2. The limit of economic haul is reached when the cost of haul and the cost of excavation become equal.

   a. Cheaper to waste the cut.

   b. The limit of economic haul will depend upon:

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
(1) Nature of the terrain.		
(2) Availability of equipment.		
(3) Type material.		
(4) Accessibility.		
(5) Availability of manpower.		
(6) Etc.		
3. Free-haul Distance - A distance over which it is considered that haul involves no extra cost.		
a. Usually about 500 feet.		
b. Limits of economic haul need to be considered for longer hauls.		
4. Procedures/steps in making a mass diagram.	II.E.4 Pass out SCBT 410.2 EA IS 1.2.9.3 Mass Diagram.	
a. Table of cumulative yardage.		
b. Under end areas put the cross section area at each station.	II.E.4.a. Refer to information sheet.	
(1) Cut, fill or both.		
c. Under volumes put the volumes of cut and /or fill between stations.		
d. Besides the sections at each full station, sections are taken at every plus station.		

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

(1) Cut and fill equal zero.

e. Cut volumes are designated as plus and fill volumes are designated as minus.

f. Under "algebraic sums volumes, cumulative" put the cumulative volumes at each station and each plus, computed in each case by determining the algebraic sum of the volume at that station or plus and the preceeding cumulative total.

5. Plotting the Mass Diagram.

   a. The vertical coordinates are cumulative volumes, plus or minus from line of zero yardage.

   b. Each horizontal line represents an increment of cubic yards.

   c. The horizontal coordinates are the stations.

   d. Each vertical line representing a full 100 foot station.

   e. Changes from cut to fill correspond to a maximum in the mass diagram curve.

   f. Changes from fill to cut correspond to a minimum.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
III. Application		
A. Discussion	III.A. Questions	III.A. Answers
	1. What is grade used for?	1. To control and give the construction effort.
	2. Name the three methods discussed which are used to determine end areas.	2. Geometric, mechanical and stripper method.
	3. How is volume determined using the average end area method?	3. Average of the two end areas and multiply by the distance between them.
	4. Shrinkage can vary from what to what?	4. 10 to 30 percent.
	5. What is free-haul?	5. Distance over which haul envolves no extra cost.
B. Practical Performance	III.B. Read and explain criterion test requirements to students and have them perform.	III.B. Perform the requirements of the criterion test.
1. Establish grade.		
2. Draw finish cross-sections and profile.		
3. Compute earthwork.		
4. Draw mass diagram.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
IV. Summary		
A. Grade.		
B. Earthwork Computations.		
C. Volume Determinations.		
D. Net Volumes by using earthwork factors.		
E. Mass Diagram.		
V. Test		
A. None		

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INFORMATION SHEET

TITLE: Volume Determination.

EXAMPLE PROBLEM: I- the following table of level notes the road bed is 20 ft. wide with slopes of 1:5:1

STATION	CROSS SECTION			AREA SQ.FT.	VOL.CU.YD.
	Lt. Side	C	Rt. Side		
115+00	C 4.0	C6.0	C 12.0	212	575
	16.0	0	28.0		
116+00	C 2.0	C 3.0	C 8.0	103	
	13.0	0	22.0		
MID -	C 3.0	C 4.5	C 10.0	154	
SECTION	14.5	0	25.0		

In the prismodial formula.

$V = \frac{100}{6}$ (212 = 4 x 154 + 103) = 15,520 cu. ft. or 575 cu. yd.

Computed by $V = \frac{(A_1 = A_2)}{54}$ L = 583 cu. yd 583 - 575 = 8 cu.yd.

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INFORMATION SHEET

TITLE: Example Converted Soil Volumes.

EXAMPLES:

SOIL TYPE	SOIL CONDITION INITIALLY	CONVERTED TO		
		IN PLACE	LOOSE	COMPACTED
Sand	In-place	1.00	1.11	0.95
	Loose	0.90	1.00	0.86
	Compacted	1.05	1.17	1.00
Loam	In-place	1.00	1.25	0.90
	Loose	0.80	1.00	0.72
	Compacted	1.11	1.39	1.00
Clay	In-place	1.00	1.43	0.90
	Loose	0.70	1.00	0.63
	Compacted	1.11	1.59	1.11
Rock (Blasted)	In-Place	1.00	1.50	1.30
	Loose	0.67	1.00	0.87
	Compacted	0.77	1.15	1.00
Hard Coral	In-place	1.00	1.50	1.30
	Loose	0.67	1.00	0.87
	Compacted	0.77	1.15	1.00

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INFORMATION SHEET

TITLE: Mass Diagram

TABLE OF CUMULATIVE YARDAGE

	End Areas (ft)		Volume (yd)		Algebraic Sums
STATION	Cut	Fill	Cut	Fill	Volumes, Cumulative
0+00	186	0			0
1+00	65	0	+465		+465
2+00	44	0	+202		+667
3+00	22	0	+122		+789
3+50	0	0	+20		+809
4+00	0	22		-20	+789
5+00	0	44		-122	+667
6+00	0	65		-202	+465
7+00	0	186		-465	0
8+00	0	119		-563	-563
9+00	35	46	+65	-305	-803
9+08	0	0	+5	-7	-805
10+00	0	22		-37	-842
10+50	0	0		-20	-862
11+00	22	0	+20		-842
12+00	44	0	+122		-720
13+00	87	0	+242		-478
14+00	215	43	+563	-80	+5
15+00	64	22	+521	-120	+406
15+07	0	0	+8	-8	+406
16+00	32	0	+55		+461
16+50	0	0	+30		+491
17+00	0	32		-30	+461
18+00	0	61		-172	+289
19+00	0	157		-405	-116
20+00	90	95	+166	-466	-416

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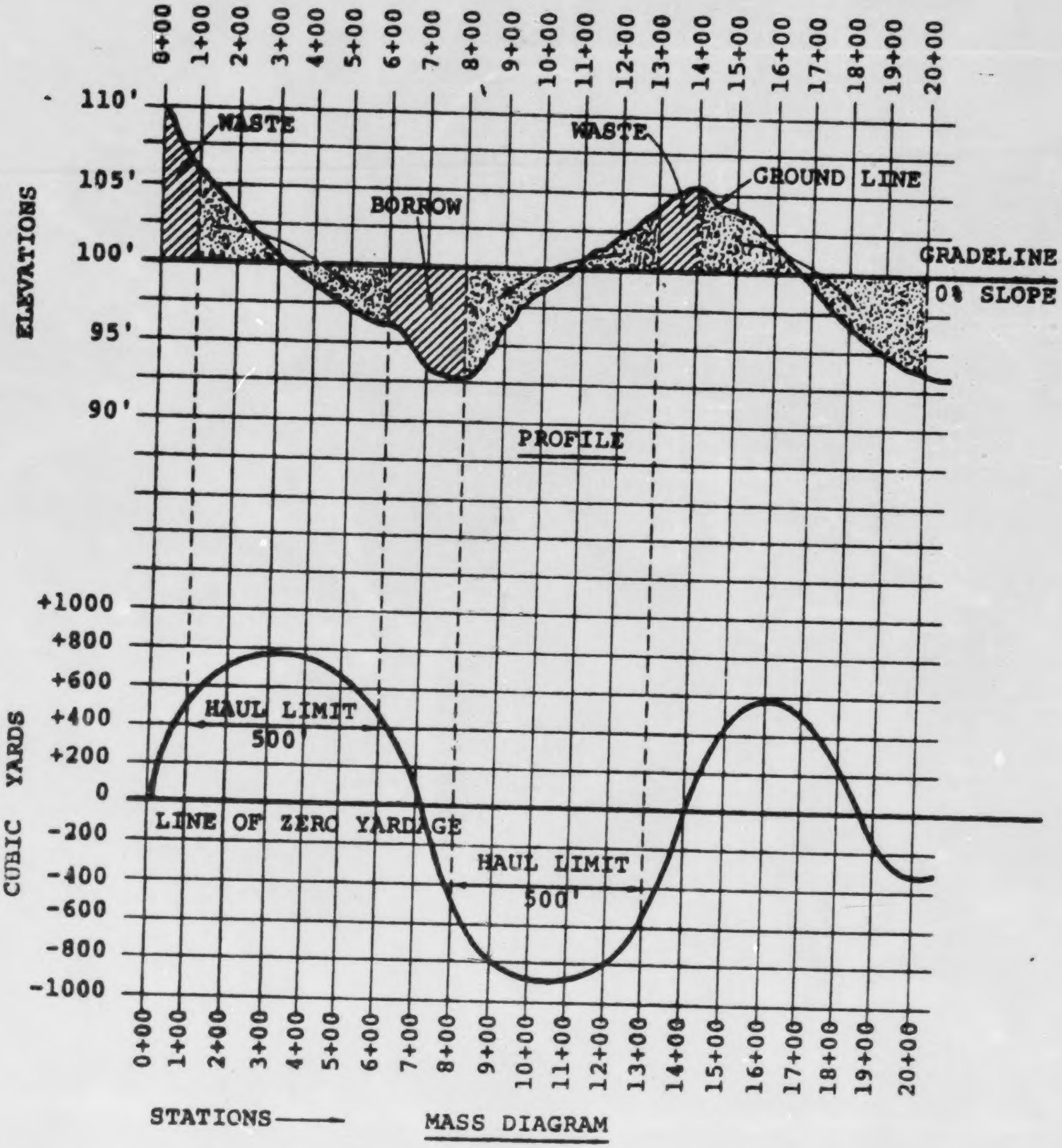
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9+00
10+00
11+00
12+00
13+00
14+00
15+00
16+00
17+00
18+00
19+00
20+00
ELEVATIONS
110'
105'
100'
95'
90'
WASTE
BORROW
WASTE
GROUND LINE
GRADELINE
0% SLOPE
PROFILE
CUBIC YARDS
+1000
+800
+600
+400
+200
0
-200
-400
-600
-800
-1000
HAUL LIMIT
500'
LINE OF ZERO YARDAGE
HAUL LIMIT
500'
STATIONS
MASS DIAGRAM

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Engineer's Transit Adjustment

Average Time: 2 Periods (Class) 3 Periods (Pract)

Instructional Materials:

A. Texts:

   1. Engineering Aid 3 & 2, NAVPERS 10634-B

B. References:

   1. Surveying Practice, Philip Kissam.

C. Tools, Equipment and Materials:

   1. Films: None

   2. Transparencies:

      a. 12-11017.1T-22, Two Peg Test.

      b. 12-11017.1T-31, Different Types of Crosshairs.

   3. Charts: None

   4. Locally Prepared Material.

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will be able to perform the proper sequence of transit adjustment and test in the field. The student reference materials and guidance of the instructor may be utilized in the performance of the requirements of the topics found in the SCBT 410.2 EA JS 1.2.10.1. Students will test each adjustable transit part until such errors are neutralized, and will be capable of attaining a third order accuracy when used in survey.

Criterion Test: Upon completion of this topic each student will be able to adjust all transit parts in sequence and test each part adjusted until each error is neutralized. Such test will make the transit adjusted capable of attaining at least a third order accuracy.

Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B, pp 352-364.

a. Job Sheet

(1) SCBT 410.2 EA JS 1.2.10.1,
Transit Adjustment

5. Devices.

a. Cutaway Transit.

E. Training Aids Equipment.

1. Overhead Projector.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Engineer's Transit Adjustment.		
B. Establish readiness.	I.B. Motivate studnets.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of materials being presented.	
1. Value		
a. Importance of surveying .		
b. Get advanced.		
c. Pass course		
d. Be a better Engineering Aid.		
D. Overview.	I.D. State learning objectives.	
1. You will be able to perform the proper sequence of transit adjustment and test in the field.	I.D.1. State information and materials necessary to guide students.	
2. Ask questions.		
3. Take notes.		
4. Testable.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
II. Presentation.		
A. Adjustments of the transit (test & correct).	II.A. Make certain that instruments are clean before making adjustments.	III.A.
1. Plate bubbles.		
a. Object - to make the plate bubbles center when the vertical axis is vertical.	II.A.1. Stress proper adjustment sequence.	
b. Test.		
c. Adjustment.		
d. Repeat test.		
e. Neutralization.		
f. Geometry.		
2. Line of sight.		
a. Object - to make the line of sight perpendicular to the horizontal axis.		
b. Test.		
c. Adjustement.		
d. Repeat test.		
e. Neutralization.		
f. Geometry.		

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OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

3. Horizontal Axis.

    a. Object - to make the horizontal axis perpendicular to the vertical axis.

    b. Test.

    c. Adjustment.

    d. Repeat test.

    e. Neutralization.

    f. Geometry.

4. Telescope level bubble.

    a. Object - to make the telescope level bubble center when the line of sight is horizontal. The procedure is often known as the peg adjustment.

    b. Test.

    c. Adjustment.

    d. Repeat test.

    e. Neutralization.

    f. Geometry (see Fig. C-11, pp309 by Philip Kissman).

5. Vertical circle (arc).

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
a. Object - to make the vertical circle indicate zero when the line of sight is perpendicular to the vertical axis.		
b. Test.		
c. Adjustment.		
d. Neutralization.		
e. Geometry.		
III. Application.		
A. Discussion.	III.A. Questions	III.A. Answers
B. Practical Performance	III.B. Instruct students in the preparation to work Job Sheet SCBT EA JS 1.2.10.1	III.B. Work Job Sheet.
1. Adjust transit.		
IV. Summary.	IV. Stress the importance of knowing how to adjust the transit when on as NMCB deployment.	
A. Plate bubbles.		
B. Line of sight		
C. Horizontal Axis.		
D. Telescope level bubble.		
E. Vertical Axis.		

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NAVAL CONSTRUCTION TRAINING CENTER
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SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

JOB SHEET

Topic: Transit Adjustment.

Introduction: The purpose of this job sheet is to guide you in the proper procedure of adjusting the transit.

Tools, Equipment and Materials:

1. Transit with tripod.
2. Philadelphia rod.
3. Plumb bob.
4. Steel tape.
5. Magnifying glass.
6. Adjusting pin.
7. Stakes/hubs.
8. Sledge hammer.
9. Field book/pencil.

Procedures:

1. Instructor will organize each survey crew composing of 3 to 4 men.
2. Instructor will demonstrate proper sequence of adjustment.
3. Instructor will assign each survey crew an area of operation.
4. Students will check and adjust transit by following the proper sequence of adjustments in accordance to the job sheet.
5. Instructor will observe the students on how they apply the procedures indicated in the job sheet and test the instrument if it is in fact in proper adjustment.

I. Plate Level Adjustment.

A. Purpose.

1. To make the axis of each plate level lie in a plane perpendicular to the vertical axis.

B. Test.

1. Set up instruments.

2. Center bubbles precisely.

3. Turn 180° in aximuth.

4. The bubble should center.

C. Correction.

1. Bring bubble halfway to center with leveling screws.

2. Bring bubbles to center with capstan screw.

II. Horizontal Crosshair.

A. Purpose.

1. To center the crosshair on the optical axis.

2. (CAUTION) This adjustment should not be made if it can be avoided, as it needs to be only approximately correct and it disturbs three other adjustments.

B. Test.

1. If crosshairs appear to be in the center of the field of view, they are near enough to the optical axis to give good results.

2. Run this test only if crosshairs appear to be far out from the center of the field of view.

a. Check transit for level.

(1) Over point "A".

b. Set stake "B" 5 to 10 feet from "A".

c. Set stake "C" at least 300 feet from "A"

- d. Read level rod.
    - (1) First on "B"
    - (2) Then on "C".
- e. Plunge the telescope.
- f. Sight on "B".
    - (1) With both axis clamped.
- g. Read rod on "C".
    - (1) With both axis clamped.
- h. Any difference is double the error.

III. Adjustment of the vertical crosshair (preliminary).

- A. Purpose.
    - 1. To make the line of sight perpendicular to the horizontal axis.
- B. Test.
    - 1. Sight a well defined point.
    - 2. Move line of sight up and down.
        - a. With tangent screw.
    - 3. Vertical crosshair should remain on point.
- C. Correction.
    - 1. Loosen two adjacent reticule capstan screws.
    - 2. Gently tap sides of screws.
        - a. Until crosshair rotates to correct position.
    - 3. Tighten the same screws.

IV. Adjustment of vertical crosshair (final).

- A. Purpose.
    - 1. To make the line of sight perpendicular to the horizontal axis.
- B. Test.

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1. Check transit for level.
2. Backsight carefully on well defined point.
   a. 200 ft. or more away.
3. Clamp plates.
4. Plunge the telescope.
5. Set foresight "A".
   a. 200 feet or more (Approx. same distance as step 2.)
6. Turn approx. 180° to the right.
7. Sight original point again.
8. Plunge the telescope.
9. Vertical hair should drop on same point as before (or point "b").

B. Correction.

1. Loosen the top reticule capstan screw.
2. Loosen one side screw, tighten opposite screw.
3. Move vertical hair 1/4 the error.
   a. Toward foresight point (from "B" to "A".)
4. Repeat test and correction until error is adjusted.

V. Elevation Axis (Standards)

A. Purpose.

1. To make the Horizontal axis perpendicular to the vertical axis.

B. Test.

1. Level transit carefully.
2. Sight a well defined point "A".
   a. At least a 30° angle (Vertical).

3. Clamp the plates.

4. Depress the telescope.

5. Set a point "B" on or near the ground.

6. Loosen lower clamp.

7. Rotate the instrument.

    a. On vertical axis.

8. Sight point again.

    a. Clamp the plate.

9. Depress the telescope again.

10. Set another point "C".

    a. Along side "B".

11. Any difference is double the error.

C. Correction.

1. Set a point "D".

    a. 1/4 distance from "B" to "C".

2. Raise or lower horizontal axis.

    a. Bring vertical hair to "D".

3. Repeat test and correction until reading is same.

VI. Telescope level.

A. Purpose.

1. To make the telescope level when the line of sight is horizontal.

B. Test.

1. Same as two peg method for level.

C. Correction.

1. Set crosshair to correct reading on rod.

    a. Use the telescope tangent screw.

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2. Center the level bubble.

    a. Use the capstan screw.

3. Repeat test and correction until readings are same.

Vii. Vertical Circle Vernier.

A. Purpose.

1. To make the vertical circle vernier read zero, when the line of sight is perpendicular to the vertical axis.

B. Test.

1. Center both plate bubbles.

2. Center telescope bubble.

3. Read vertical vernier.

4. Angle read is index error.

C. Correction.

1. Loosen capstan screws.

2. Tighten same screws after moving.

VIII. Repeat all adjustments.

A. In proper sequence.

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NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Level Adjustments

Average Time: 2 Period (Class) 3 Period (Pract)

Instructional Materials:

A. Texr.

1. Engineering Aid 3 & 2, NAVPERS 10634-B

B. References:

1. Surveying Practice, Philip Kissam.

C. Tools, Equipment and Material:

1. See Annex III.

D. Training Aids and Devices:

1. None

E. Training Aids Equipment:

1. None

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will be able to adjust the dumpy, wye and self leveling levels. Procedure for testing and adjusting levels shall be sat forth by this instructor's guide and Engineering Aid 3 & 2, NAVPERS 10634-B. Students shall test and adjust the levels until all errors are neutralized and the instrument is capable of attaining 3rd order accuracy when used in surveying operations.

Criterion Test: Each student will test and adjust a level (dumpy, wye or self-leveling) following the procedure established by this instructor's guide and Engineering Aid 3 & 2, NAVPERS 10634-B. Students shall test and adjust the levels until all errors are neutralized and the instrument is capable of attaining 3rd order accuracy when used in surveying operations.

Homework: Read

Engineering Aid 3 & 2, Chapter 10, pp. 359 thru 362.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name		
2. Topic: Level Adjustment.		
B. Establish readiness.	I.B. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview.	I.D. State learning objectives.	
1. You will be able to adjust levels.	I.D.1. State information and material necessary to guide student.	
2. Ask questions.		
3. Take Notes.		
4. Testable.		

II. Presentation.

A. Field Adjustments to the Dumpy and Wye Level.

1. General procedures:

   a. Test frequently but adjust rarely.

   b. Be positive the problem is with the instrument and not with the deficiencies in the test.

2. To test the instrument properly, observe the following precautions:

   a. Choose a cloudy day when possible.

   b. See that the tripod shoes are tight and the instrument is firmly screwed on the tripod.

   c. Set up the tripod on firm ground, out of the sun, with legs spread well apart.

   d. Make certain that the hinge screws are not too loose to allow freedom of movement nor too tight to cause residual friction.

   e. Attach the sunshade and carefully focus the eyepiece.

   f. Go through all the tests in the order given. Do not adjust the instrument unless a particular test indicates the same amount of error at least three times.

3. After any adjustment has been made, the proper test should be applied at once.

4. After all the contemplated adjustments have been completed, all of the tests should be applied again in the proper order, in case some other adjustments might have been disturbed.

B. Test and Procedures for Adjustment of the Dumpy Level.

1. Parallax.

a. Sight through the telescope.

(1) Make preliminary focus of eyepiece on cross-lines.

(2) Turn knurled eyepiece until wires appear sharp and black.

b. Focus.

(1) On clearly defined point.

(2) Well lighted.

(3) 300 feet away.

(4) Turn focusing knob.

(a) Backward and forward.

(b) Slowly.

(c) At the same time wagging head.

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

(5) Observe for apparent lateral movement between target image and cross-lines.

(6) Stop focusing at point where no lateral displacement appears.

(7) Disregard sharpness of image and of cross-lines.

(8) This objective focusing is important in the elimination of parallax.

c. Sharpen image (if necessary).

(1) Refocus eyepiece slightly.

(2) Cross-lines will be more distinct.

d. Refocusing.

(1) Tired eyes.

(2) Different observer.

(3) Repeat steps "b" and "c".

e. High order surveys.

(1) Step "b" should be followed on all pointings.

(2) Eliminates parallax due to improper focusing.

2. To make the axis of the bubble perpendicular to the vertical axis or spindle.

   a. Test.

      (1) Set up level on tripod.

         (a) Bring telescope over two diagonally opposite leveling screws.

         (b) Bring bubble to center of tube.

      (2) Rotate level about spindle 180 degrees.

         (a) Note whether bubble remains in center of tube.

   b. Adjustment.

      (1) Bring bubble half-way back to center with leveling screws.

      (2) Correct balance of error with capstan nuts at either end of bubble tube.

      (3) Alternate over both pairs of leveling screws until bubble remains in center of tube when rotated about spindle.

3. To make the horizontal cross-line perpendicular to the vertical axis or spindle.

   a. Test (see Fig. 410.2-3-1-1)

      (1) Set up level on tripod.

         (a) Set one end of horizontal cross-line on sharply defined point.

      (2) Turn level slowly about spindle.

         (a) Use slow motion tangent screw.

         (b) Horizontal line traces over point.

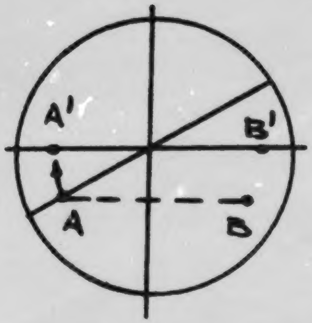
         (c) If line coincides with point throughout, position is correct.

   b. Adjustment.

      (1) Slightly loosen all four reticle capstan screws.

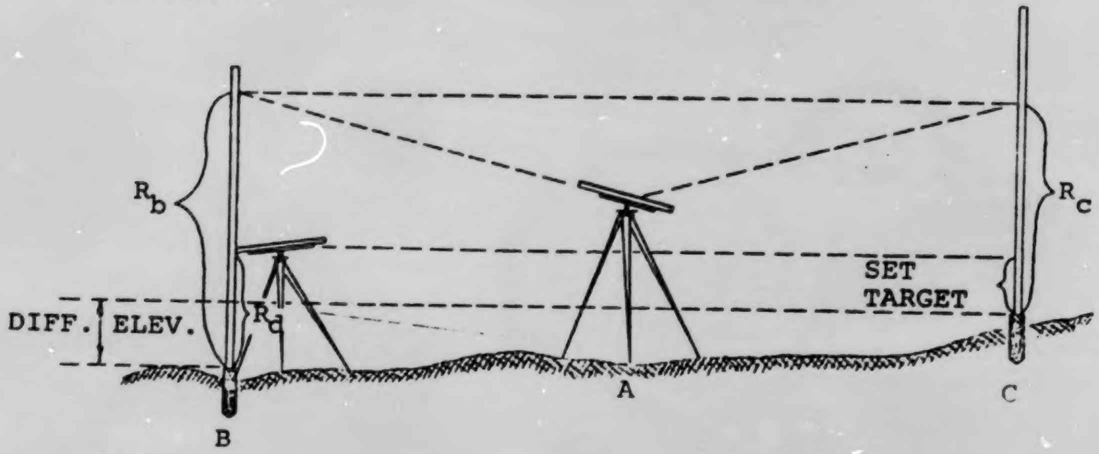
      (2) Move cross-line ring around in proper direction until horizontal cross-line exactly traces (A' B')

      (3) Tighten reticle capstan screw and rerun test.



4. To make the line of sight parallel to the axis of the bubble (the "Two Peg Method").

   a. Test



(1) Set up level at some convenient point "A".

  (a) Holding rod at "C".

  (b) Distance at 100 feet.

  (c) Instrument carefully leveled.

  (d) Read rod on "C".

  (e) Call reading "R_c".

(2) Locate point "B".

  (a) Directly behind instrument.

  (b) Distance "AB" equal to "AC".

(3) Point telescope toward "B".

  (a) Bring bubble to center of telescope tube.

  (b) Take rod reading "R_b".

(4) Set up level beside point "B".

  (a) Eyepiece of telescope directly over a point.

  (b) Level up carefully

(5) Point eyepiece of telescope toward rod at "B".

(a) Read rod through objective end of telescope.

(b) Call rod reading "R_d".

(c) If more convenient measure along outside center line of telescope.

(6) Add to "R_d" the difference between the first readings, ($R_c - R_d$).

b. Adjustment.

(1) Set rod target to result of test (6).

(a) Hold rod on point "C".

(2) Move cross-line ring up or down.

(a) Set exactly on target.

(b) Turning vertical pair of opposite capstan screws.

(3) Check again.

(a) Read rod on "B".

(b) Compute rod reading for "C".

(c) Observe whether horizontal cross-line cuts target.

5. To Make Cross-lines Appear in Center of Field.

   a. Test.

      (1) Notice whether cross-lines are in center of field.

         (a) Note whether lines appear right, left, above, or below center.

   b. Adjustment.

      (1) Mark ring just forward of focusing ring with pencil.

         (a) Place mark at top.

         (b) Use mark to position eye piece.

      (2) Remove eyepiece from telescope.

         (a) Note four brass centering screws in the body.

         (b) Use pencil mark to indicate direction.

         (c) If cross-lines appear to the left under test (1).

            1. Loosen screw at extreme left of pencil mark about 1/4 turn.

            2. Tighten screw to the right the same amount.

(d) If opposite movement is required, reverse procedure.

(e) If cross-lines must be lowered.

1. Loosen screw near pencil mark 1/4 turn.

2. Tighten bottom screw by same amount.

(f) If opposite movement is required, reverse procedure.

(3) Replace eyepiece.

(a) Refocus cross-lines.

(b) Repeat adjustments (1) and (2) as required.

C. Test and Procedures for Adjustment of the Wye Level.

1. Parallax (same as for the dumpy levels).

2. To make the axis of the bubble parallel to and in the same vertical plane with the axis of the wye rings.

a. Test

(1) Hold level sideways with spindle horizontal and turn focusing screw until level balances.

(a) Set up the level on tripod.

(b) Clamp telescope over two diagonally opposite leveling screws.

(2) Remove wye pins and raise wye clips.

(3) Bring bubble to center of tube.

(4) Lift telescope out of wyes.

(a) Turn end for end.

(b) Replace in wyes.

(c) Note whether bubble remains in center of tube.

b. Adjustment.

(1) Bring bubble halfway back to center by leveling screws.

(2) Correct balance of error by turning capstan nuts at eyepiece end of bubble tube.

c. Test.

(1) Rotate telescope in wyes.

(a) About 30 degrees either side of vertical.

(b) Note whether bubble remains in center of tube.

d. Adjustment.

(1) Bring bubble all the way back to center by turning lateral capstan screws on each side of the bubble tube post at objective end of level.

(2) Repeat test a (3) and (4) and adjustment "b" (1) and (2).

(3) Check alternately until both the lateral adjustment and the vertical adjustment of vial are correct.

3. To make the axis of the wyes perpendicular to the vertical axis or spindle.

a. Test.

(1) Set up level.

(a) Rotate telescope about spindle over two diagonally opposite leveling screws.

(b) Bring bubble to center of tube.

(c) Check telescope bubble adjustment.

(d) Telescope slide must be in position of balance.

(2) Rotate level about spindle 180 degrees.

(a) Note whether bubble remains in center of tube.

b. Adjustment.

(1) Bring bubble half-way back to center by leveling screws.

(a) Raise or lower one end of wye bar.

(b) Bring bubble to center.

(c) Turning a pair of capstan nuts at either end of wye bar.

(2) Repeat until bubble remains in center of tube when rotated about spindle.

4. To make the horizontal cross-line perpendicular to the vertical axis or spindle.

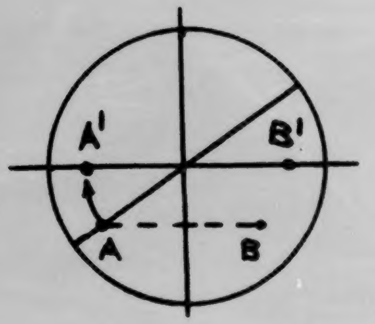
a. Test.

(1) Set up level on tripod.

(a) Set one end of horizontal line on sharply defined point "A".

(b) Horizontal line traces over point.

(c) If line coincides with point throughout, position is correct.



b. Adjustment

(1) If point appears to trace line "AB".

(a) Release pressure on reticle capstan screws slightly.

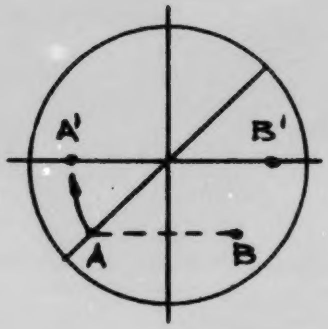
(b) Turn all four capstan screws only slightly and by equal amounts.

(2) Gently tap capstan screws.

(a) In direction to close angle between horizontal line and dotted line "AB".

(b) Rotate cross-line ring until horizontal line exactly traces point from A' to B'.

(3) Tighten capstan screws (all four equally) and check.



5. To make the line of sight (collimation) pass through the axis of the wye rings.

a. Test.

(1) Set up level on tripod.

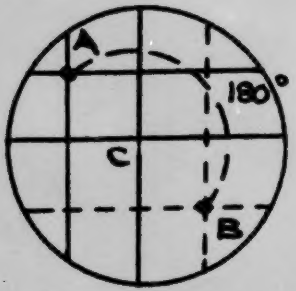
(a) Remove wye pens from clips.

(b) Rise clips so that telescope is free to rotate.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY

(2) Set intersection of cross-lines on well defined point A.

(a) Figure



(b) About 300 feet away.

(3) Carefully rotate telescope half-way around its wyes.

(a) Note whether intersection of cross-lines still covers point.

b. Adjustment.

(1) Move the telescope.

(a) By leveling and tangent screws

(b) Until error seems one-half corrected.

(2) Move cross-line ring.

(a) Using each pair of opposite capstan screws successively.

(b) Until error is entirely corrected.

(c) Cross-line intersection now covers point "C".

(3) Repeat rectification and collimation of the cross-lines until both adjustments are correct. (Adjustment #4).

6. To Make Cross-lines appear in center of Field (same as dumpy level).

D. Care of Self-Leveling Level.

1. The initial care of the self-leveling level is similar to the engineer's level as concerns the inspection, trying the motions, wiping the metal surfaces and cleaning the lenses.

2. Care during operations.

a. Keep level in box when not in use and when transporting to and from working area.

b. Raise telescope off micrometer screw whenever the instrument is moved. 421

420 c. Set the clamp screw to prevent the instrument from swinging, but not overtightened.

d. Keep lenses and working parts clean at all times.

e. Avoid sunlight effects.

(1) Always use umbrella for shade.

(2) Cloudiness is not protection from the uneven expansion and contraction of the instrument.

E. Field Adjustments of the Self-Leveling Level.

1. General procedures:

a. Test frequently but adjust rarely.

b. Be positive the problem is with the instrument and not with deficiencies in the test.

c. To test the instrument properly, observe the following precautions:

(1) Choose a cloudy day when possible.

(2) Use an umbrella to shade the instrument.

(3) See that the tripod shoes are tight and the instrument is firmly screwed on the tripod.

(4) Set up the tripod on firm ground out of sun, with the legs spread apart.

(5) Make certain that the hinge screws are not too loose to allow freedom of movement nor too tight to cause residual friction.

(6) Attach the sunshine and carefully focus the eyepiece.

(7) Go through all the tests in the order given. Do not adjust the instrument unless a particular test indicates the same amount of error at least three times.

d. After any adjustment has been made, the proper test should be applied at once.

e. After all the contemplated adjustments have been made, all the tests should be applied again in the proper order, in case some other adjustments might have been disturbed.

2. Procedures for adjustments to the self-leveling level.

a. Adjustment to the circular level to vertical axis.

(1) Center circular level accurately with leveling screws.

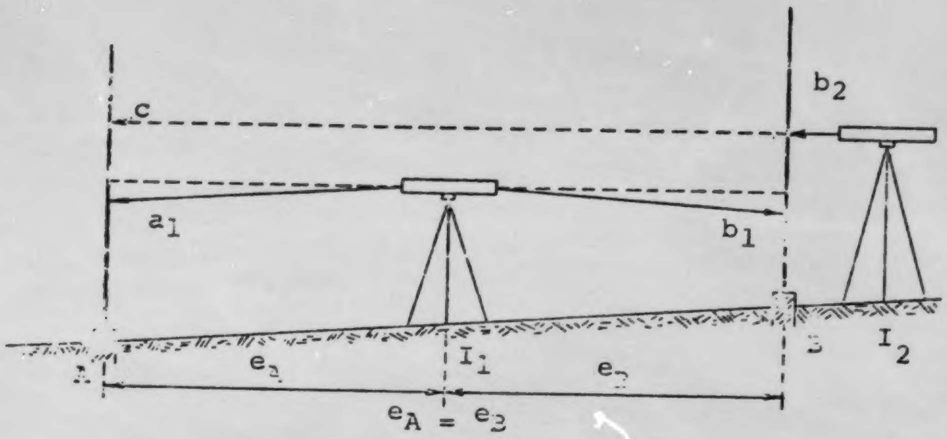
(2) Reverse telescope through 180 degrees (Approx.)

(3) Eliminate half the bubble error by means of the leveling screws and remaining half with adjusting screws. Before tightening an adjustment screw, release the one on opposite side accordingly.

(4) Reverse telescope back into original position. The bubble should now remain central. In case of a residual error of more than 0.2 mm. Repeat procedure as described in (c). All adjusting screws should be tightened moderately.

b. Adjustment of the line of collimation to truly horizontal.

(1) Adjust by leveling from center position. (See fig.) set up instrument in the center between two staves approximately 200 feet apart. Center the adjusted circular level and read both staves (a_1 and b_1 see fig.) The difference in elevation between A and B is $a_1 - b_1 = d$ and corresponds to the true difference even when the line of sight is not in adjustment.



$a_1 = 2.423$
$-b_1 = 0.936$
$d = 1.487$

$b_2 = 1.462$
$+d = 1.487$
$c = 2.949$

(2) Set up self-leveling level at a distance of 3.5m. (12 feet) in the front or behind the higher staff (B). Center circular level and take reading b_2 then the reading on staff a should have the value C.

(3) If the actual reading differs from the predetermined c reading by more than 2mm. (0.01 feet) then the crossline has to be adjusted so that they do match. The capstan screw for the crossline setting is visible when cover ring is removed.

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(4) Repeat the procedure as a check.

(5) If only one staff is available the positions A and B must be well marked and the staff must be set up carefully.

c. Adjustment of the lateral fine setting screw.

(1) The motion of the lateral fine setting screw can be adjusted by counterlocking or loosening the knurled controls.

d. Adjustment of the friction clutch for rough setting.

(1) The stiffness of the friction movement for the rough setting of the instrument is adjustable by means of a disc situated above the circle setting disc.

e. Adjustment of the leveling screw movement.

(1) The motion of the leveling screws should not be altered. If it is found necessary, however, an adjustment can be made with a screwdriver on the screws situated on the inside above the knurl of the leveling screws.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
III. **Application.**		
A. Discussion	III.A. Questions.	III.A. Answers
	1. The motion of the lateral fine setting screw can be adjusted by	1. Counterlocking or loosening the knurled controls.
	2. What method is used to adjust the line of collication to truly horizontal in the self leveling level?	2. Two peg test method.
B. Practical Performance:	III.B. Read and explain criterion test, have student perform same.	III.B. Perform the requirements of the criterion test.
1. Student will check and adjust class instruments as appropriate.		

IV. Summary

A. Field adjustment to the dumpy and wye level.

B. Test for adjustment of the dumpy level.

C. Test for adjustment of the wye level.

D. Care of self-leveling level.

E. Field adjustment of the self-leveling level.

V. Test.

A. None

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NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified.

Topic: Utility Surveying.

Average Time: 2 Period (Class) 5 Period (Pract)

Instructional Materials:

A. Texts.

1. Engineering Aid 3 & 2, NAVPERS 10634-B, Chapter 16.

B. References:

1. Engineering Surveys; 2nd Edition, Rubey, Lommel and Todd; Chapter 13.

C. Tools Equipment and Materials:

1. None

D. Training Aids and Devices:

1. None

E. Training Aids Equipment:

1. None

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. Each student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will be able, while acting as a member of a survey party, perform a utility survey, staking out a gravity flow sewer line. Procedures will be as outlined in this topic, Engineering Aid 3 & 2, NAVPERS 10634-B. Student work will be accurate to 0.01 of a foot per 25 foot station.

Criterion Test: Each student will do a utility survey, while acting as a member of a survey party, involving a section of gravity flow sewer line of at least 300 feet in length. Student work will be accurate to 0.01 of a foot per 25 foot station.

Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B, Chapter 16, pp. 531-532.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.		
A. Establish contact.	I.A. Introduce self and topic.	
1. Name.		
2. Topic: Utility Surveying.		
B. Establish readiness.	I.B. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass couse.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to perform a utility survey staking out a gravity flow sewer line.	I.D.1. State information and materials necessary to guide student.	
2. Take notes.		
3. Ask questions.		
4. Testable.		

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

II. Presentation.

A. Utility surveys.

1. General term applied to:

a. Pipe lines (such as sewer, water, gas and oil pipe lines).

b. Communication lines (such as telephone).

c. Electrical power lines.

B. Above ground utilities.

1. Usually consists of communication and electrical pole lines.

2. Distribution lines (service).

a. Location is usually controlled by the building it serves.

b. Usually follows streets and roads.

3. Transmission lines (to distribution).

a. Judgment in route selections is required.

4. Survey consists simply of locating the horizontal line as prescribed and marking stations.

a. Often guys and anchors can be located also.

b. Sometimes pole height for vertical clearance obstructions is determined.

C. Underground utilities.

1. Often need both lines and grade.

2. Pressure lines (such as water).

a. Usually only necessary to stake out line because the only grade requirement is soil cover.

b. May require elevation if area is to be graded first.

c. May require elevation if conflicting underground utilities will be or have been constructed.

3. Gravity flow lines (such as storm sewers).

a. Requires staking for grade as well as line.

4. Grade, given as invert or flow line.

a. Invert-bottom inside of a drainage channel or pipe.

b. Flow line-bottom inside of drainage pipe.

c. Inlet and outlet.

(1) Outlet controlling elevation for manhole.

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OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

D. Procedure for location and construction layout.

1. Establish controlling points.

   a. Connecting points.

      (1) Man holes, changes in direction or grade.

      (2) Houses.

      (3) Connections to existing lines.

2. Centerline stakeout.

   a. Mark centerline stations.

      (1) 1" x 2' stakes used.

      (2) 25 or 50 ft. spacing normal.

3. Offset line.

   a. Right or left of centerline.

   b. Distance determined by type of equipment used to dig trench.

   c. Mark offset stake with 2" x 2" hub and tack.

   d. Set guard stakes.

      (1) Mark guard stakes w/following.

(a) Front of stake (side to centerline.)

1. Set grade stake or mark at each end of the uniform grade.

2. Cut (to nearest 1/8") hundreth.

(b) Steps.

1. Set grade stake or mark at each end of the uniform grade.

2. A transit or level is then set up over one end.

3. Difference in height between the instrument and the mark is measured and the target on the rod is set at this value (grade rod).

4. Rod held at other end on slope and line of sight directed at it.

5. Lock in line of sight. You can now set a given mark on the line or sight anywhere desired.

(2) Giving grade.

(a) Most rapid and in many ways the best method.

OUTLINE OF INSTRUCTION **INSTRUCTOR ACTIVITY** **STUDENT ACTIVITY**

(b) Indicates cut from a convenient object near work.

(c) Elevation is determining on convenient objects or stakes by profile leveling.

1. Difference computed and stakes or batter boards marked.

2. Keel usually marks point to which measurement is to be taken.

(3) Offset distance.

(a) Rear of stake (side away from centerline).

1. Station No.

2. Direction from centerline (right to left).

e. Points on streets and pavements.

(1) Mark with chisel, spikes or paint.

4. Profile run.

a. Determine ground elevation to nearest 0.1 ft. at centerline stations.

b. Determine elevations on offset hubs to nearest 0.01 ft.

5. Plot profile.
    - a. Use appropriate horizontal and vertical scales.
6. Compute rate of grade (gradient).
    - a. Formula:

$$\frac{\text{Difference in Elev.}}{\text{Hor. Dist.}} \times 100 = \%\ \text{grade}$$

    - b. Methods of giving grade.
        - (1) Shooting in grade-used when marks are to be set for a uniform rate of grade.
            - (a) Not an independent method.
7. Set batter boards.
    - a. Convenient locations.
    - b. Invert of pipe used for controlling grade.
8. Keep appropriate notes and records of any construction change.
    - a. Used to make "as builts".
    - b. Sample on page 212,"Surveying Practice".

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
III. Application		
A. Discussion	III.A. Questions	III.A. Answers
	1. How is location and design determined on a sewer line?	1. Starting and ending points.
	2. Why is an offset line used?	2. Centerline will be destroyed during construction.
	3. What is an invert?	3. Bottom, inside of drainage channel.
B. Practical Application.	III.B. Discuss problem with students. Have students set up field notes grade requirements per station.	III.B. Students perform required survey work.
IV. Summary.		
A. Utilities surveys.		
1. General.		
B. Above ground utilities.		
1. Consists of.		
2. Distribution.		
3. Transmission.		
4. Survey consists of.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
C. Underground utilities.		
1. Requirements.		
2. Pressure lines.		
3. Gravity flow lines.		
4. Grade.		
D. Procedure for location and construction layout.		
1. Establish control.		
2. Centerline.		
3. Offset.		
4. Profile run.		
5. Plot profile.		
6. Compute rate of grade.		
7. Set batter boards.		
8. Notes.		
V. Test.		
A. See criterion test.		

NAVAL CONSTRUCTION TRAINING CENTER
PORT HUENEME, CALIFORNIA 93043
SPECIAL CONSTRUCTION BATTALION TRAINING (SCBT) 410.2

Classification: Unclassified

Topic: Building Layout.

Average Time: 2 Periods (Class) 8 Periods (Pract)

Instructional Materials:

A. **Texts:**

1. Engineering Aid 3 & 2, NAVPERS 10634-B.

B. **References:**

1. Surveying Practices, Philip Kissum.

C. Tools, Equipment and Materials:

1. See Annex III.

D. Training Aids and Devices:

1. Films: None
2. Transparencies: None
3. Charts: None
4. Locally Prepared Materials:

Terminal Objective: Upon completion of this unit each student will have performed engineering surveys related to area and route surveying. The student will have acted as a member of a survey party, performing the duties of each member, meeting the requirements of this unit. Conditions and standards are as set forth in individual topic objectives.

Enabling Objectives: Upon completion of this topic each student will be able to layout a married pre-engineered steel building (2-40' x 100') while acting as a chainman, rodman, instrumentman and notekeeper in a survey party. Procedures will be as outlined in Engineering Aid 3 & 2, NAVPERS, 10634-B, Chapter 16, Page 532 through 534 and Manufacturer's Drawings and Specifications. Student application will be checked by measuring the diagonals with an accuracy of ± 1/16 inch as measured on the diagon and checked against each other and the computed diagonal distance.

Criterion Test: Each student will act as a rodman, chain man, notekeeper and instrumentman while laying out two married pre-engineered steel buildings with an accuracy of ± 1/16 inch as measured on the diagonals and checked against each other and the computed diagonal distance.

Homework: Read

Engineering Aid 3 & 2, NAVPERS 10634-B, pp. 532 - 534.

a. Data Sheets.

(1) SCBT 410.2 EA DS 1.2.13.1,
Sample Note Format Building Layout.

E. Training Aids Equipemnt:

1. None

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
I. Introduction to the lesson.	I.A. Introduce self and topic.	
A. Establish contact.		
1. Name.		
2. Topic: Building Layout.		
B. Establish readiness:	I.B. Motivate student.	
1. Purpose.		
2. Assignment.		
C. Establish effect.	I.C. Bring out need and value of material being presented.	
1. Value.		
a. Pass course.		
b. Perform better on the job.		
c. Get advanced.		
d. Be a better Engineering Aid.		
D. Overview:	I.D. State learning objectives.	
1. You will be able to layout a married pre-engineered steel building following manufacturer's specifications.	I.D.1. State information and materials necessary to guide student.	
2. Take notes.		
3. Ask questions.		
4. Testable.		

OUTLINE OF INSTRUCTION INSTRUCTOR ACTIVITY STUDENT ACTIVITY

II. Presentation.

A. Construction Survey.

1. Procedures of properly positioning field construction is termed layout.

2. May be a simple building foundation or the intricate arrangement of interrelated buildings, utility lines, streets, roads, and other facilities.

3. Data for survey.

a. Plans always give either by scale or by actual dimensions the positions and elevations of the new work.

(1) Relative to existing structures or to survey control marks.

(2) The dimensions of the construction shown on the plans complete the necessary data for giving line and grade.

b. Sometimes construction plans are not available and you have only manufacturer's erection plans and spec's.

(1) You must establish your own control and site plan.

(2) Draw a rough floor or foundation plan for use in laying out the construction site.

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
B. Location and layout procedures.		
1. Location and control		
a. Determine boundries of site and verify or mark.		
b. Establish reference line (base line).	II.B.1.b. Demonstrate procedures.	
(1) Away from the construction area.		
c. Establish TBM and reference.	II.B.1.c. Demonstrate procedures.	
(1) Away from the construction area.		
2. Layout.		
a. Set building corners.		
(1) Can be located in several ways.		
(a) Two distances from known points.		
(b) Angle and distance from known points.		
(c) Angles from two known points.		
(d) Perpendicular swing offset from known base line (most used).	II.B.2.a.(1)(d) Demonstrate each step.	
1. Transit and steel tape.		
2. Mark corners with hub and tack.		

OUTLINE OF INSTRUCTION | **INSTRUCTOR ACTIVITY** | **STUDENT ACTIVITY**

3. Establish guard stakes and mark.

3. Check accuracy of rectangular layout.

   a. Small buildings.

      (1) Make angular measurements at each corner.

      (2) Measure diagonals.

   b. Large buildings.

      (1) Angular measurements at each corner.

4. Batter boards.

   a. Temporary device which supports cords which:

      (1) Mark outline of building.

      (2) Grade for structure.

   b. Constructed of 2" x 4" stakes and 1" x 6" cross pieces (Surveying Practices, page 207).

   c. Procedure for setting.

      (1) Set 2" x 4" stakes.

         (a) Back 3 or 4 feet from outer edge of excavation line.

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OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
(b) Mark to grade.		
1. Finish floor grade.		
2. Even number of feet above or below finish grade.		
(c) Nail top of crosspiece flush with marks.		
(d) Mark top of crosspiece with nail or notch on building line.		
5. Notekeeping.	II.B.5. Pass out SCBT 410.2 EA DS 1.2.10.1	
6. Continuing work.		
a. Reference used to re-establish.		
(1) Footing and foundation line.		
(2) Plumb forms.		
(3) Columns, steel frame work etc.		
b. References may be used to establish reference points within a building for locating piping, machinery or interior structural features.		
(1) This could also include a bench mark.		
7. As builts.		

OUTLINE OF INSTRUCTION	INSTRUCTOR ACTIVITY	STUDENT ACTIVITY
III. Application.		
A. Discussion.	III.A. Questions.	III.A. Answers
	1. How would the plans give data necessary for building layout?	1. By scale or by actual dimension.
	2. Which of the four methods of layout is preferred?	2. Perpendicular offset.
	3. How would you check the accuracy of a small building layout?	3. Diagonals & angles.
	4. What distance should a batter board be offset?	4. 3 to 15 feet or out of the way of construction.
B. Practical performance.	III.B. Pass out a set of manufacturer's plans and specifications and have each party draw out a rough foundation plan. Explain layout problem to students with demonstration.	III.B. Perform buildng layout.
IV. Summary		
A. Construction Survey.		
B. Location and layout procedures.		
1. Location and control.		

OUTLINE OF INSTRUCTION | INSTRUCTOR ACTIVITY | STUDENT ACTIVITY

2. Layout.

3. Check accuracy.

4. Batter boards.

5. Note keeping.

6. Continuing work.

7. As builts.

V. Test.

A. See criterion test.

SAMPLE NOTE FORMAT - BLDG. LAYOUT

SCBT 410.2 EA DS 1.2.13.1

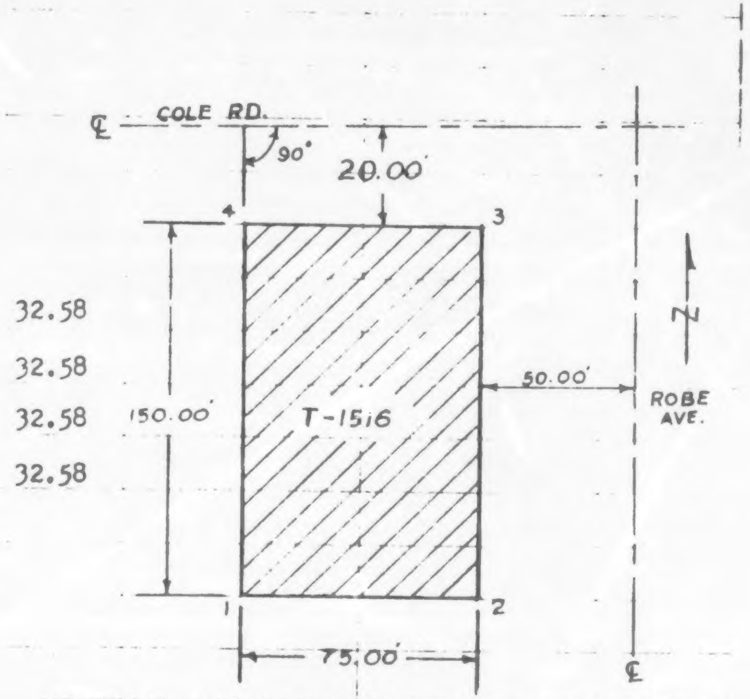
BUILDING LAYOUT
BLDG T-1516, PORHUE, CA.

1 APR 67
FAIR, WINDY (52°)
K&E LEVEL # 451206
K&E 1' TRANSIT 4219

⊼ SNIPES
▣ TILKIN
ø LUFKIN
ø COWART

10

STA.	BS	HI	FS	ELEV.	GRADE ROD	BATTER ELEV.	REMARKS
BM 17	5.22	35.22		30.00			
1			4.26	30.96			
2			4.14	31.08			
3			4.68	30.54			
4			4.52	30.70			
1					2.64	32.58	
2					2.64	32.58	
3					2.64	32.58	
4					2.64	32.58	



NOTE: BATTERBOARD ELEVATION 1.5' ABOVE HIGHEST CORNER.
BATTERBOARD ELEVATION IS TOP OF FOUNDATION

NAVPERS 10634-B

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3-27

CHAPTER 10

ADJUSTMENT AND REPLACEMENT OF SURVEYING EQUIPMENT

As used in surveying, the term ADJUSTMENT has two meanings, depending on its usage and intent. When applied to the instrument, adjustment means bringing the various fixed parts into proper relation with one another, so accurate results are obtained when the instrument is used. It is distinguished from the ordinary operations of leveling the instrument, aligning the telescope, and so forth. When the term is used in connection with the results of a survey, adjustment may mean one of the following: (1) The proper distribution of errors of closure in a closed traverse, (2) the angular adjustment of a station and/or figure in a triangulation network, and (3) the adjustment of elevations in a level circuit.

As used in this chapter, adjustment is considered in terms of the first definition and information is presented on minor adjustments and repairs of surveying instruments and the replacement of equipment. The adjustment of survey results is discussed fully in Engineering Aid 1 & C.

MINOR ADJUSTMENTS

The minor adjustments, minor repairs, and the replacement of surveying instruments are among the responsibilities of engineering aid personnel. Minor adjustments and minor repairs are those that can generally be done in the field using simple tools; major adjustments and repairs are those generally done in the factory. If the defect in the instrument cannot be corrected by minor adjustment or repair, do not attempt to disassemble it; make necessary arrangements for sending the instrument to the manufacturer. Most surveying instruments are precision instruments; major adjustments and recalibration need special skills and tools that can be provided only by the instrument company or its subsidiaries.

The EA should be familiar with the minor adjustments that are considered in this chapter. These adjustments are not laborious, nor are the basis of the adjustment principles difficult to understand. In order to make proper adjustments, it is important that the EA take the following into consideration:

1. He must be familiar with the principles upon which the adjustments are based.
2. He must know the methods or tests used to determine if an instrument is out of adjustment.
3. He must know the procedure for making actual adjustments, and the correct sequence by which they can be made expeditiously.
4. He must be able to tell what effect the adjustment of one part will have on other parts of the instrument.
5. He must understand the effect of each adjustment upon the instrument when it is actually used for measurement.

Generally, the adjustments of surveying instruments involve the level bubbles, telescope, and the reticle. For example, if one or both of the plate level bubbles of an engineer's transit are centered when the plate is, in fact, not level, the instrument is out of adjustment. Similarly, an optical instrument equipped with vertical and horizontal crosshairs is out of adjustment if the point of intersection between the crosshairs does not coincide with the "optical axis". Similarly again: if the reflected bubble on a Locke or Abney level is centered when the optical axis is other than horizontal, the instrument is out of adjustment.

The process of adjustment involves chiefly the steps which are necessary to bring a bubble to center when it should be at center, or to bring a crosshair point of intersection into coincidence with the optical axis. Instrument manufacturers publish handbooks containing recommended adjustment procedures. These are usually small pamphlets, obtainable free of charge. The Keuffel & Esser Solar Ephemeris, an annual publication consisting chiefly of astronomical tables, is such a pamphlet. Besides the tables, the Solar Ephemeris contains detailed adjustment procedures for K & E transits, levels, and alidades.

The following sections, taken from the K & E Solar Ephemeris, are intended to give you an idea of general instrument adjustment procedures. For adjusting your particular instruments, you should follow the appropriate manufacturer's instructions.

GENERAL ADJUSTMENT PROCEDURE

Instruments should be carefully checked periodically to determine whether or not they need adjustment. There is an adage that an instrument should be checked frequently but adjusted rarely. The basis for this adage is the fact that modern quality instruments get out of adjustment much less frequently than is generally believed. Consequently, much need for adjustment is caused by previously made improper adjustments which were not really required, but resulted from errors in checking.

Before it is assumed, then, that adjustment is necessary, it must be positively ascertained that an apparent maladjustment is actually such, and not a result of error in the check, or of circumstances other than maladjustment. The following procedures should be followed in checking.

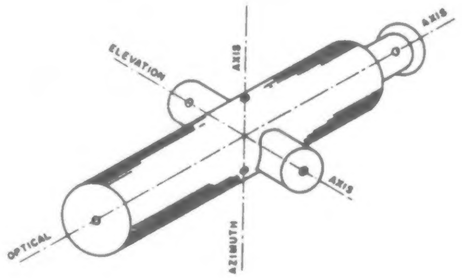
1. Check on a cloudy day, if possible.
2. Ascertain that the tripod shoes are tight and that the instrument is screwed all the way down on the tripod.
3. Set up on firm ground, in shade but in a good light, where a sight of at least 200 ft can be taken in opposite directions.
4. Spread the tripod feet well apart and place them so as to bring the plate approximately level. Press the shoes in firmly, or set them in cracks or chipped depressions if on a hardened surface. (Avoid setup on asphalt pavement in warm weather.)
5. After the tripod feet are set, release and then retighten the wing nuts. The purpose of this is to release any possible "residual friction" which, if not released, might cause an eventual shift in the legs.
6. Level the instrument with particular care. After leveling, loosen all level screws slightly (again to release residual friction) and relevel. Tighten all screws with equal firmness, but avoid tightening too tight. Too much tightness will eventually deform the centers, causing both friction and play.
7. Carry out all checks in the order prescribed for the instrument. Do not make an adjustment unless and until the same check, repeated at least 3 times, indicates the same amount of error every time.
8. Remember that most tests show an error which is DOUBLE the actual displacement error in the instrument.

Be especially watchful for "creep"—that is, a change in position caused by settlement or by temperature change in the instrument. To detect any possible creep, allow every set bubble or set line of sight to stand for a few seconds, and ensure that no movement occurs during the interval.

Before making an adjustment, consider whether or not the error discovered will have a material effect on field results. Make adjustments in prescribed order, to avoid disturbing an adjustment by a subsequent adjustment. After an adjustment is made, set up the parts firmly but not too tight. Then repeat the original check at once. After all the contemplated adjustments have been made, repeat the entire round of checks in the prescribed order. This will indicate whether or not an adjustment has been disturbed by a subsequent adjustment.

TRANSIT ADJUSTMENTS

Figure 10-1 illustrates the meanings of the terms "azimuth axis", "opitcal axis", and "elevation axis" as they apply to a transit, engineer's level, or alidade.



45.434

Figure 10-1.—Principal axes of surveying instruments, such as the transit, engineer's level, or the alidade.

The transit must be kept in good adjustment to obtain accurate results. There are six tests and adjustments of the transit that you must be capable of performing. All tests and adjustments of the transit are made with the instrument mounted on its tripod and set up in the shade. These tests should be made periodically, in the sequence in which they are discussed in the following paragraphs. When one of the tests indicates that an adjustment is necessary, this adjustment must be made and all previous tests must be repeated before proceeding with the next test.

Plate Bubbles Adjustment

The test of the plate bubbles should be made every time the instrument is set up for use. When an error in the adjustment of either plate level is indicated, it is not always necessary to make the adjustment, because the operation of bringing the bubble halfway back to center by turning the leveling screws makes the vertical axis of the transit vertical. The adjustment must be made, however, before other tests and adjustment of the instrument. To make the axis of the bubbles perpendicular to the vertical axis (fig. 10-2), perform the following steps:

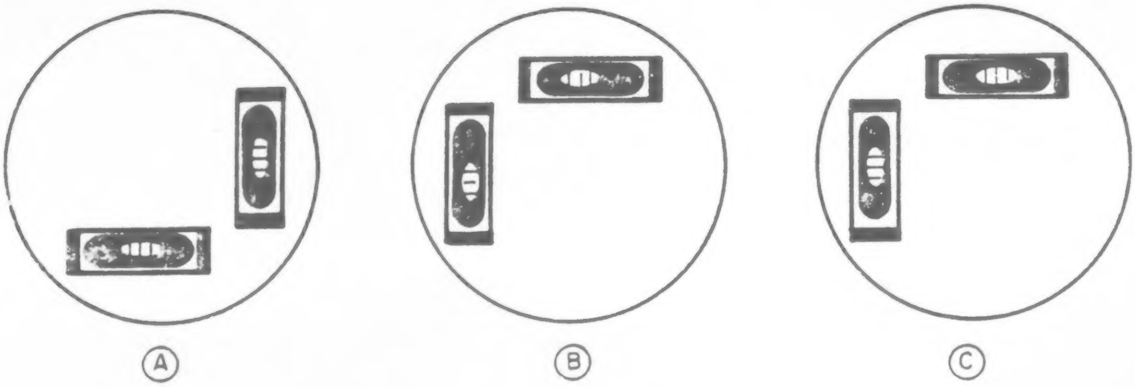
1. Set up the transit and bring both bubbles to the center of their tubes by turning the leveling screws (view A, fig. 10-2).
2. Rotate the instrument about its vertical axis through 180° and note the amount the bubbles move away from their center (view B, fig. 10-2).
3. Bring the bubble of each tube half the distance back to the center of its tube by turning the capstan screws at the end of each tube.
4. Relevel with the leveling screws and rotate the instrument again. Make similar correction if the bubbles do not remain in the center of the tubes.
5. Check the final adjustment by noting that the bubbles remain in the center of the tubes during the entire revolution about the vertical axis (view C, fig. 10-2).

Crosshair Reticle Adjustment

The point of intersection between the horizontal and the vertical crosshairs should lie on the optical axis. The optical axis is at the center of the circular field of view through the telescope.

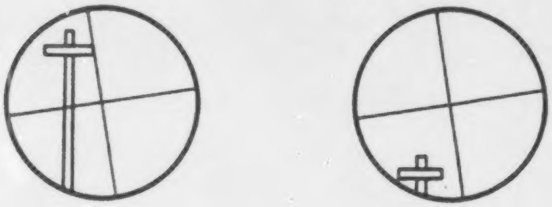
To make the vertical crosshair lie in a plane perpendicular to the horizontal axis (fig. 10-3), follow the procedure below:

1. See that parallax is eliminated. Sight the vertical crosshair on a well-defined point, and with all motions clamped, move the telescope slightly up and down on its horizontal axis, using the vertical slow motion tangent screw. If the instrument is in adjustment, the vertical wire will appear to stay on the point through its entire length.
2. If it does not, loosen the two capstan screws holding the crosshairs and slightly rotate the ring by tapping the screws lightly.



45.751

Figure 10-2. —Adjustment of plate bubbles.



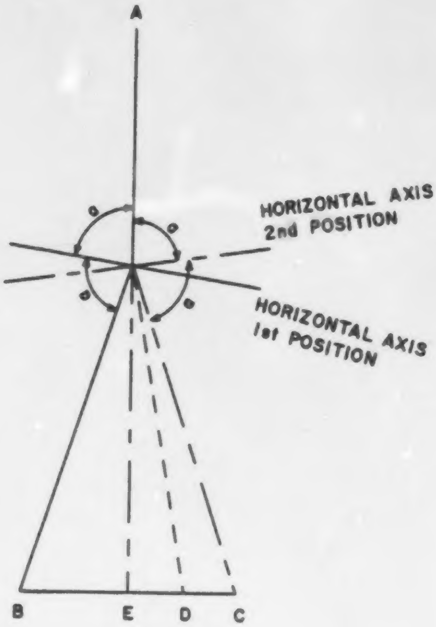
45.752
Figure 10-3.—Adjustment of the vertical crosshair.

3. Sight again on the point. If the vertical crosshair does not stay on the point through its entire length as the telescope is moved up and down, rotate the ring again.

4. Repeat this process until the condition is satisfied.

To make the line of sight perpendicular to the horizontal axis (fig. 10-4), proceed as follows:

1. Sight on a point A at a distance of not less than 200 feet with the telescope normal; clamp both plates.
2. Plunge the telescope and set another point B on the ground at a distance from the instrument equal to the first distance, and at about the same elevation as point A.
3. Unclamp the upper motion, rotate the instrument about its vertical axis, and sight on the first point (telescope inverted) and clamp.
4. Plunge the telescope and observe the second point. If the instrument is in adjustment, the point over which it is set will be on a straight line AE, and point B will fall at position E. If the instrument is not in adjustment, the intersection of the crosshairs (point C) will fall to one side of the second point B.
5. Measure the distance BC and place a point D one-fourth of this distance back toward the original point B.
6. Move the crosshair reticle horizontally by loosening the screws on one side of the telescope tube and tightening the opposite screw until the vertical crosshair appears to have moved from C to the corrected position D.
7. Repeat this operation from No. 1 above, until no error is observed.
8. Repeat the test described for adjusting the vertical crosshair, since the vertical crosshair may have rotated during this adjustment.

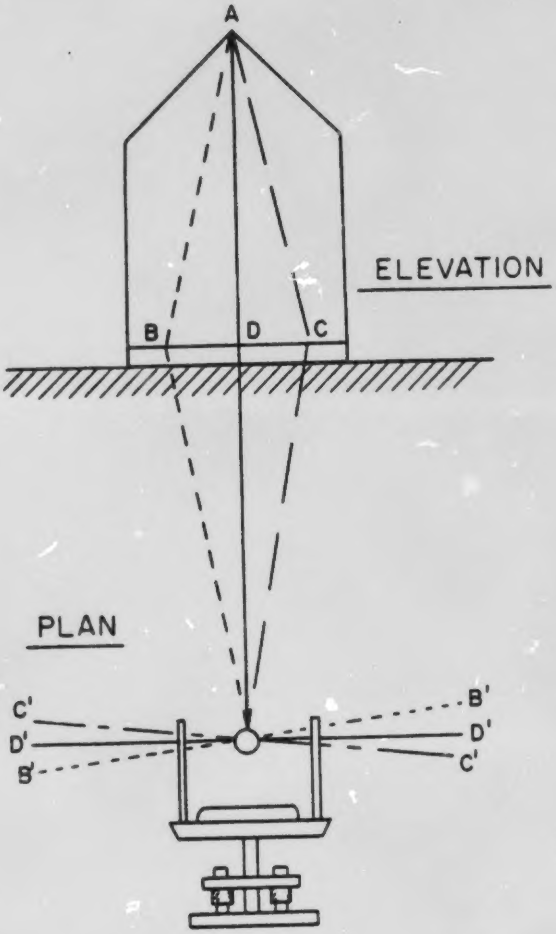


115.75(45B)A
Figure 10-4.—Adjustment of the crosshair reticle.

Elevation Axis Adjustment

To make the elevation (also called horizontal) axis of the telescope perpendicular to the azimuth (also called vertical) axis of the instrument (fig. 10-5), perform the following steps:

1. Sight with the vertical crosshair on some high point A, at least 30° above the horizontal and at a distance of 200 ft, such as the corner of the eaves of a stable building or other well-defined objects, and clamp the plates.
2. Depress the telescope and mark a second point B at about the same level as the telescope.
3. Plunge the telescope, unclamp the lower plate, and rotate the instrument about its vertical axis.
4. Sight on the first point A.
5. Clamp the vertical axis and depress the telescope. If the vertical crosshair intersects the second or lower point B, the horizontal axis is in adjustment. In this case, point B is coincident with point D in both direct and reverse positions of the telescope.



45.754
Figure 10-5.—Adjustment of the elevation axis.

6. If not, mark the new point C on this line and note the distance BC between this point and the original point.

7. Mark point D exactly midway of the distance BC. CD is the amount of correction to be made.

8. Adjust by turning the small capstan screw in the adjustable bearing at one end of the horizontal axis to correct the error.

9. Repeat this test until the vertical crosshair passes through the high and low points in the direct and inverted position of the telescope.

10. Check all previous adjustments.

Telescope Level Adjustment

The adjustment of the telescope level bubble of the transit is generally called the peg method. (This method is explained fully in the adjustment of an engineer's level which appears later in this chapter.) The steps for performing this adjustment follow:

1. The instrument is set up midway between two stakes driven 200 to 300 feet apart.

2. A reading is taken through the telescope on a rod held on each of the stakes. The telescope must be carefully leveled before each reading. The difference between readings is the difference in elevation between the stakes.

3. The instrument is moved, set up, and leveled close to one of the stakes. The eyepiece should swing within about half-inch of the rod.

4. The near rod is read through the objective, and the far rod in a normal manner leveling the telescope carefully before each reading. The difference between rod readings should equal the difference from No. 2 above, if the instrument is in adjustment. If not, a correction must be made.

5. To adjust, compute the reading that should be made on the far rod. This equals the near rod reading plus the difference from No. 2 above.

6. Set the horizontal crosshair on the computed reading using the slow motion screw, and move one end of the spirit level vertically by means of the adjusting nuts until the bubble is in the center of the tube (fig. 10-6).

Vertical Circle Vernier Adjustment

To index the vertical circle vernier to read zero when the instrument is leveled (fig. 10-7), perform the following:

1. Bring the telescope bubble to the center of the tube.

2. Read the vernier of the vertical circle.

3. If it does not read zero, loosen the capstan screws holding the vernier and move the index until it reads zero on the vertical circle.

4. Tighten the screws and read the vernier with all the bubbles in the center of their tubes to make sure that it has not moved during the operation.

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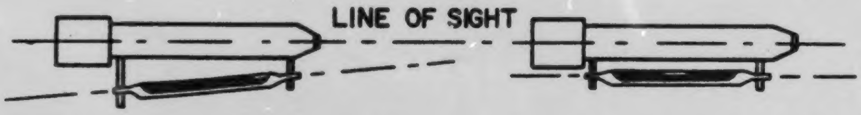
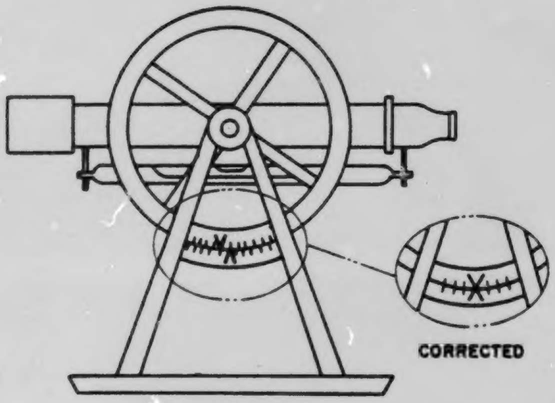


45.755
Figure 10-6.—Adjustment of telescope bubble.



45.756
Figure 10-7.—Adjustment of vertical circle vernier.

ALIDADE ADJUSTMENTS

The adjustments of an alidade is similar to that of a transit. The telescopic alidade also requires six adjustments. They should be made in the listed sequence. The seventh adjustment given here, is the only one required for the self-indexing alidade. Prior to the alidade adjustment, the plane table is set up and carefully leveled.

Crosshair Reticle Adjustments

The following procedure is used to make the line of sight through the crosshair intersection coincide with the axis of the telescope (collimation adjustment):

1. Point the alidade at a distant well-defined point.
2. Rotate the telescope in its sleeve. The intersection of the crosshairs should remain on the distant point. If the distant point appears to move away from either or both the crosshairs, they should be adjusted.
3. Adjust each crosshair separately until the intersection of the crosshairs continually bisects the distant point as the telescope is rotated through 180°.

Vertical Crosshair Adjustment

To make the vertical crosshair perpendicular to the horizontal axis of the telescope, perform the following steps:

1. See that parallax is eliminated and that the alidade is leveled. Sight the vertical hair on a well-defined point, and move the telescope slightly up and down on its horizontal axis with the slow motion screw. If the instrument is in adjustment, the vertical crosshair will appear to follow the point through its entire length.
2. If it does not, loosen the screws holding the crosshairs and slightly rotate the ring by tapping the screws lightly.
3. Sight again on the point and if the vertical crosshair does not follow the point through its entire length as the telescope is moved up and down, rotate the ring again.
4. Repeat this process until the condition is satisfied.
5. Repeat the collimation adjustment check above; to make sure that the crosshair intersection still coincides with the axis of the telescope.

Striding Level Adjustment

The following procedure is used to make the axis of the striding level parallel to the line of sight:

1. Clip the striding level into place on the telescope.
2. Center the level bubble using the tangent screw.
3. Unclip, reverse, and reclip the striding level.
4. If the bubble is off center, bring it halfway back using the tangent screw.

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5. Complete the centering, using the pair of capstan screws at one end of the bubble tube.

6. Repeat the test and adjustment until a reversal of the striding level does not move the bubble off center.

Vertical Arc Control Level Adjustment

To make the vertical arc read true vertical angles, perform the following:

1. Place the alidade on a stable, flat, approximately level surface.
2. Place the striding level on the telescope.
3. Center the bubble of the striding level in its vial.
4. Move the zero graduation of the vernier into coincidence with the 30° graduation of the vertical arc. If the bubble of the vertical arc control level comes to rest off center, use the adjusting screws near one end of the vertical arc control level to move the bubble until it is centered in the vial.

Circular Level Adjustment

Circular level adjustment is made by the following procedure:

1. Set up and approximately level the planetable.
2. Place the alidade near the center of the drawing board.
3. Draw a line along the length of the alidade blade.
4. Turn the alidade 180° and replace the edge of the blade on the line previously drawn on the board. The bubble of the circular level should now come to rest at the center of the circle.
5. If the bubble comes to rest off center, the blade must be checked for flatness. When the test indicates the blade is warped, the blade must be flattened. If a test of the level still indicates an error, the bubble should be adjusted by placing small shims under the edge of the bubble holder.

Stadia Arc Adjustment

The following procedure is used to make the stadia arc read the true stadia factors for horizontal and vertical corrections:

1. Test and adjust the vertical arc control level, as described above.
2. Inspect the stadia arc index mark or marks. The index for horizontal corrections should be in exact coincidence with the arc graduation numbered 100. The index for the vertical corrections should be in exact coincidence with the arc graduation numbered 50.
3. When the bubbles of the striding level and the vertical arc control level are both centered in their vials and the stadia arc is not properly positioned, loosen the index plate holding screws with a screwdriver, move the plate to its proper position, and clamp in place by retightening the screws.

Self-Indexing Alidade Adjustment

To set the scales of the self-indexing alidade at their correct values when the line of sight is horizontal, perform the following steps:

1. Set up and level the planetable over one of two selected points at about the same elevation and about 250 feet apart.
2. Place the rod against the planetable, slide the left side of the alidade up to the rod, and read the exact height of the friction adjusting screw on the end of the telescope axle. A pencil mark at this point on the rod will be helpful.
3. Move the rod to the other selected point, sight upon the marked point, and read the vertical angle scale.
4. Move and set up the planetable at the second position.
5. Check the height of the adjusting screw at this point and move the rod to the first point.
6. Sight on the second marked point (if not the same as the first point) and read the vertical angle scale.
7. If the instrument is in adjustment, the sum of the two readings (3 and 6 above) will equal 180°. If the sum is not 180°, the instrument needs adjustment.
8. To adjust, loosen the capstan locknut to the right of the tangent screw and move the reading an amount equal to one-half the difference between the sum and 180°.

For example:

Reading at position #1 = 89° 48'
Reading at position #2 = 90° 20'
Sum = 180° 08'

Amount of correction $= \frac{08'}{2} = 04'$

(The sum is greater than 180°, so the correction is minus.)

9. With the instrument still set up at the second position, the value is changed, 90° 20' - 04' = 90° 16'.

ENGINEER'S LEVEL ADJUSTMENTS

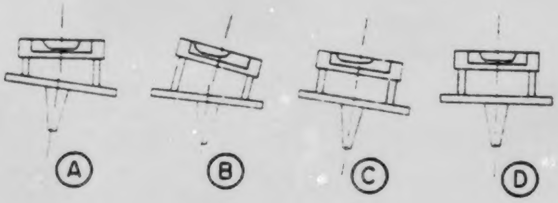
A check of the instrument's adjustment should be made upon receipt from the supplier and before it is taken to the field. It is necessary to check the adjustments every day before starting work and at any time the instrument is bumped or jolted. The instrument should be set up and approximately leveled over both pairs of screws. Since the check will also include the optical assembly, the crosshairs and objective should be focused sharply, using a well-defined object at least 250 feet away, and then the parallax removed. When parallax is present, the image is not exactly in the plane of the crosshairs and the objective focusing must be refined. Since this condition can occur each time the objective lens is focused, a parallax check must be made whenever a new object is observed.

The check and adjustment of an engineer's level is made in three steps and in the order listed.

Telescope Level Adjustments

Adjustment of the bubble tube (fig. 10-8), makes the axis of the bubble perpendicular to the axis of rotation (azimuth axis). The adjustment procedure follows:

1. Set the telescope over diametrically opposite leveling screws, and center the bubble carefully as shown in view A, figure 10-8.
2. Rotate the telescope 180° and note the movement of the bubble away from center (view B, fig. 10-8).



45.757
Figure 10-8.—Adjustment of the bubble tube.

3. Bring the bubble half the distance back to the center of the tube by turning the capstan screws at the end of the tube (view C, fig. 10-8).
4. Relevel with the leveling screws (view D, fig. 10-8) and rotate the instrument again. Repeat (3) above, if the bubble does not remain at the center of the tube.
5. Check the final adjustment by noting that the bubble remains in the center of the tube during the entire revolution about the vertical axis.

Crosshair Reticle Adjustments

The crosshairs are adjusted to make the horizontal crosshair lie in a plane perpendicular to the vertical axis. See figure 10-9. The adjustment is made by performing the following steps:

1. Level the instrument carefully.
2. Sight one end of the horizontal crosshair on a well-defined point at least 250 feet away. Turn the telescope slowly on its vertical axis, using the slow motion screw. If the crosshairs are in adjustment, the horizontal wire will stay on the point through its entire length.
3. If it does not, loosen two adjacent reticle capstan screws and rotate the reticle by lightly tapping two opposite screws.
4. Sight on the point again and if the horizontal wire does not follow the point through its entire length, rotate the ring again.
5. Repeat this process until the condition is satisfied.

Line of Sight Adjustment

The adjustment makes the line of sight parallel to the axis of the bubble tube. This is known as the "two-peg" test method (fig. 10-10). This method requires the following steps:

1. Set up the instrument (first set-up, fig. 10-10); drive stake A about 150 feet away; drive stake B at the same distance in the opposite direction. Set up the instrument so that a pair of opposite screws is parallel with line AB.
2. Take a rod reading "a" on stake A and a rod reading "b" on stake B With the instrument exactly halfway between the two stakes, (b-a) is the true difference in elevation between the stakes.
3. Move the instrument close to stake A (second set-up, fig. 10-10) so that the eyepiece swings within a half inch from the rod.

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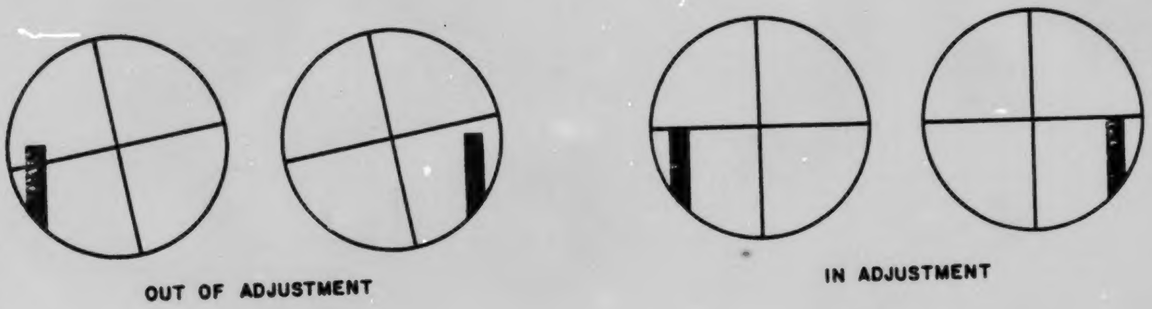
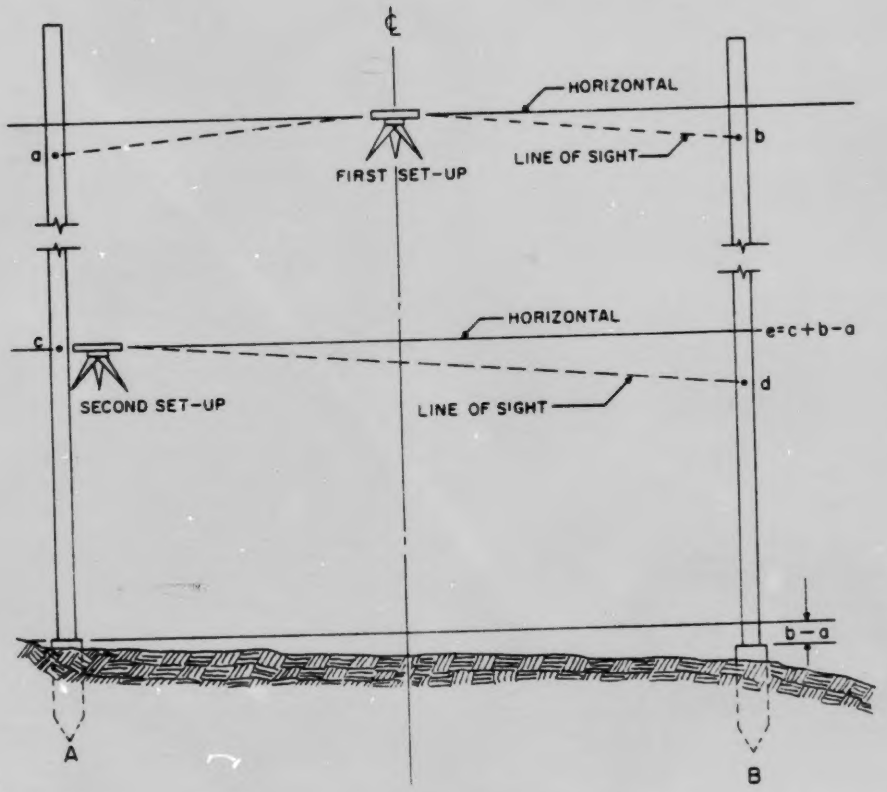


115.75(45B)B
Figure 10-9.—Adjustment of horizontal crosshairs.



45.436(45B)
Figure 10-10.—Two-peg test method.

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4. Take a rod reading "c" on stake A through the objective lens, and a rod reading "d" on stake E in the normal manner. If the instrument is in adjustment (d-c) will equal (b-a).

5. If the instrument is out of adjustment, calculate what the correct rod reading "e" should be on the farther rod B (e = c + b - a). Set rod reading e with a target for accurate reading. Move the horizontal crosshair to the correct reading (on target) by loosening the correct vertical capstan screw and tightening the opposite screw.

6. Check the horizontal crosshair adjustment again. The ring may have rotated during this adjustment.

7. Rerun the peg test to check the adjustment.

MILITARY LEVEL ADJUSTMENTS

The military level has a fixed reticle which does not permit checking or adjusting the line of sight. However, the circular level bubble and the main level bubble must be checked and adjusted. Whereas in the engineer level, the line of sight was made parallel to the bubble axis; in the military level, the bubble is adjusted to the line of sight.

Circular Bubble Adjustment

The adjustment of the circular bubble level is similar to the adjustment of the long vial bubbles which were discussed earlier in this chapter, such as the plate level bubbles of the transit. Two axes at right angles on the bubble face are assumed and each axis is treated as if it is the longitudinal axis of a plate level bubble. The capstan screws on the base of the circular bubble housing are adjusted so that the bubble will remain in the center when the instrument is turned in any direction.

Main Telescope Level Bubble Adjustment

Adjustment makes the bubble tube axis and the line of sight parallel. The two-peg method of adjustment is also used. The bubble is brought into coincidence at each pointing with the tilting screw and the rod is read. The correction is computed and the line of sight brought to the corrected reading using the tilting screw. The bubble is then adjusted roughly using the capstan-head screw at the eyepiece end of the bubble. Fine adjusting is completed by the micrometer drum under the eyepiece of the telescope. The two-peg test should be made to check the final adjustment, and must be repeated after any disturbance of the objective.

HAND LEVEL ADJUSTMENTS

Generally, a hand level is designed to stand up to rough usage without need for constant adjusting. The level vial is protected, sealed in position and kept firmly in adjustment. The tubing is seamless; the threads are accurate; the lens, the mirror prism, the level tube and the end-pieces are solidly mounted. Every part of the hand level is rigidly held in proper position; however, the level is very easy to adjust, if ever it becomes out of adjustment.

The simplest method of adjusting the hand level is by placing it alongside the engineer's level; the engineer's level is first leveled and sighted on a well-defined point. Then when this is done, the hand level is held alongside the telescope of the engineer's level as it is sighted on the point. The line of sight of the hand level should hit the same point when its bubble is centered.

If you are adjusting a Locke level, you must manipulate the screw at the end of the level tube which controls the crosshairs defining the line of sight. If it is an Abney level, you must raise or lower one end of the level tube vial until the bubble is centered; make sure that before doing this, you have first set the index to zero on the graduated arc.

A hand level that is out of adjustment may be used to establish a horizontal line by employing the principle of the two-peg test method (explained earlier in this chapter), with a little variation. Let positions A and B, be two posts, trees, corners of a building, or other convenient objects on a fiarly level ground and about 30 to 50 feet apart. Let's suppose that we selected two trees for this purpose, as shown in figure 10-11. Using a sharp knife, make a small horizontal notch C (at a convenient height) on trunk of tree A; hold the level against this notch, and with the bubble centered, establish point D—making a small notch also at this point. The level is then held at notch D and point E is established in the same manner when the level was held at notch C. The distance CE would be double the error, and point M, the midpoint between C and E will therefore represent the horizontal line through notch D. The Locke level

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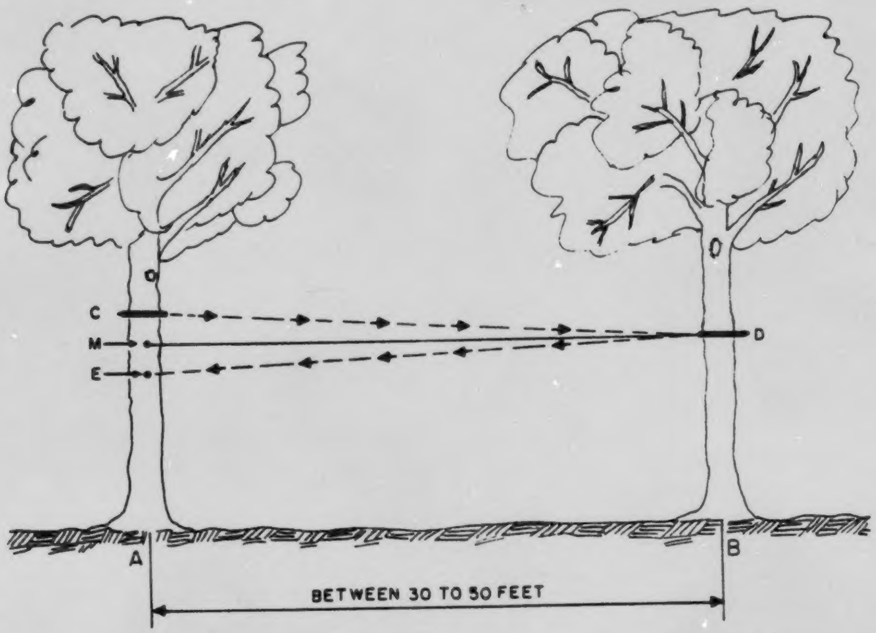

45.436(45B)
Figure 10-11.—Two-peg method of adjusting a hand level.

or the Abney level, whichever type of level you are testing is adjusted accordingly.

SEXTANT ADJUSTMENTS

Like any other surveying instruments, the sextant must be kept in good adjustment to obtain accurate results. These adjustments must be performed in the sequence in which they are discussed in this section.

Index Mirror Adjustment

To adjust the index mirror perpendicular to the plane of the instrument:

1. Set index arm near middle of arc.

2. Hold instrument with eye close to index mirror, and in plane of sextant with arc away from eye.

3. Observe arc direct and reflected in index mirror, moving index arm back and forth. Arc and reflected image of arc should appear as a continuous unbroken arc.

4. If arc appears to be broken or noncontinuous, correct by adjusting screw at back of index mirror frame.

Horizon Mirror Adjustment

To adjust the horizon mirror perpendicular to the plane of the instrument:

1. Set index near zero.
2. Hold sextant vertically and sight horizon.
3. Bring direct and reflected image of horizon into coincidence by turning micrometer.
4. Incline sextant so its plane makes a slight angle with the horizon. Images should still be in coincidence.
5. If images do not remain in coincidence, correct by adjusting horizon mirror using the screws on the back of the frame.

Index Mirror Correction

To correct index mirror by making reflecting surfaces of the two mirrors parallel when index is exactly at zero:

1. Set index at zero.

2. Point at distant object (usually the horizon).
3. Rotate horizon mirror by means of adjusting screw until direct and reflecting images coincide.

After each day's use, the sextant should be wiped clean and dried off. To prevent excessive wear on the movable parts, refrain from adjusting the sextant unnecessarily. When mirrors fog or become useless, replace them; do not throw old mirrors away. They can be resilvered and reused.

MINOR REPAIRS

As stated earlier in this chapter, minor repairs to surveying instruments and equipment are those that can be done in the field with the use of simple tools. These repairs are done by the SEABEES. Major repairs are done by instrument specialists who are generally employed by the instruments manufacturers, and the repairs are done in the factory.

Whether or not you yourself, or someone else in the battalion, should attempt the repair of a damaged item of equipment depends on the nature of the damage and the character of the item. A broken tape, for example, can easily be spliced (explained in chapter 12). On the other hand, whether or not you should attempt to straighten a bent compass needle depends on the type of compass—for an ordinary pocket compass, perhaps yes; for the compass on a transit, perhaps no. Many types of damage to such articles as range poles, tripod legs, and the like may be repaired in battalion or PWD shops. Minor damage to instruments may be repaired occasionally in battalion machine shops, but major repairs to instruments, when they are economically worth while at all, should be done by manufacturers or their authorized agents, or by competent Navy instrument repairmen.

REPLACEMENT PROCEDURE

When in the judgment of the senior EA or the Engineering Officer, an instrument is beyond economical repairs, it must be surveyed using standard survey procedures. It must be replaced with a new one, and the procurement of which will normally be carried through the Navy supply system. Expendable items are procured in the same manner.

NAVY SUPPLY SYSTEM

Each individual item of equipment or supply which is available through the Navy supply system is identified by a stock number, and listed and described in a stock catalog. The items which may be drawn from supply by a battalion, and the maximum number of such items a battalion may have, are set forth in an "allowance list". When the number of items available in a battalion falls short of the allowance (because of expenditure, wear, casualty, loss, or some other type of attrition), the shortage must be replaced.

Some items, such as range poles, sounding poles, chaining pins, bull points, turning point pins, targets, stake bags, equipment boxes, and the like, may be replaced by manufacture in battalion or Public Works Department shops. Most items, however, are replaced from supply—that is, they are ordered from the nearest available Naval Supply Depot.

An item is ordered by stock number in accordance with a prescribed procedure with which you must be familiar. Your source to study for most of what you should know about the Navy supply system is chapter 12 of Military Requirements for Petty Officers 3 & 2, NavPers 10056-B. There is an additional phase of the system, however, which is not discussed in NavPers 10056-B, and which is peculiar to the SEABEES. This is the "advanced base functional component system"

ADVANCED BASE FUNCTIONAL COMPONENT SYSTEM

During the Pacific Campaign of World War II, materials ordered for one base were appropriated for the more immediate need of another base that happened to be nearer the battlefront. At times it was impossible to know what part of, or how much of a grouping of materials that had been planned for a certain project had been diverted to another project of higher priority. To correct this shortcoming, logistics planners attempted to standardize and catalog in related groups all the motor vehicles, engineering equipment, radios, typewriters, boats, number of men, and consumable supplies from the numerous allowance lists. The problem was tackled from the point of tasks to be performed, and who and what was needed for each task. The modular or building block concept was developed, with each component designed to serve a specific function no matter where it was placed. Thus, the Functional Component System as a tool of Naval logistics evolved out of the experience with early advanced base planning and shipment of materials in World War II.

It is the result of an analysis of all foreseeable overseas requirements of men and materials and the organization of such requirements into compact and related units for assignment and shipment, individually or in multiples, to fill any sized operational requirement. The aggregate of all these units or functional components, large and small, in their relationship to each other, is the Functional Component System as developed by the U.S. Navy. All advanced base logistic planning is expressed in terms of functional components or their equivalents. The Functional Component System has become the quantitative expression of measurement of planning, procurement, assembly and shipping of material needed for war and other operational requirements. The system is intended to facilitate and expedite procurement by area commanders simply to order a component, instead of ordering each item in the component separately.

As a SEABEE, you must be familiar with NAVAL FACILITIES ENGINEERING COMMAND's Detailed Catalog of Equipment and Construction Material Requirements for Advanced Base Functional Components, NAVFAC P-103. It is in this publication that you will find the allowance list for outfitting a Mobile Construction Battalion. It is called P-25 Component. Under P-25 Component, you will find Assembly No. 4002 which lists the surveyor's supplies and equipment requirement for a 4-man survey crew. The quantities could be increased in multiples of a crew requirement which depends upon the authorized strength of the battalion, or upon the needs of the projects in a deployment.

The NAVAL CONSTRUCTION FORCES MANUAL, Annex C, gives an excellent summary description of the Functional Component System.

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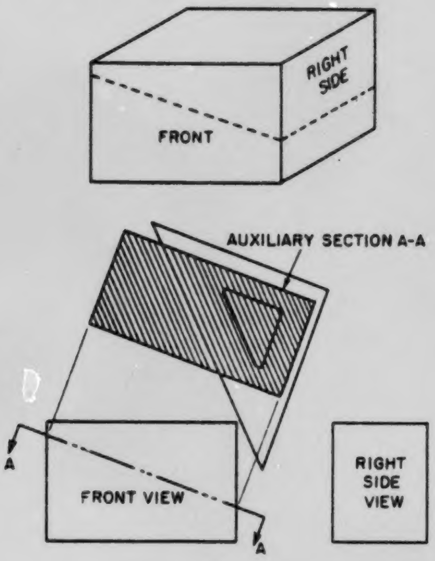


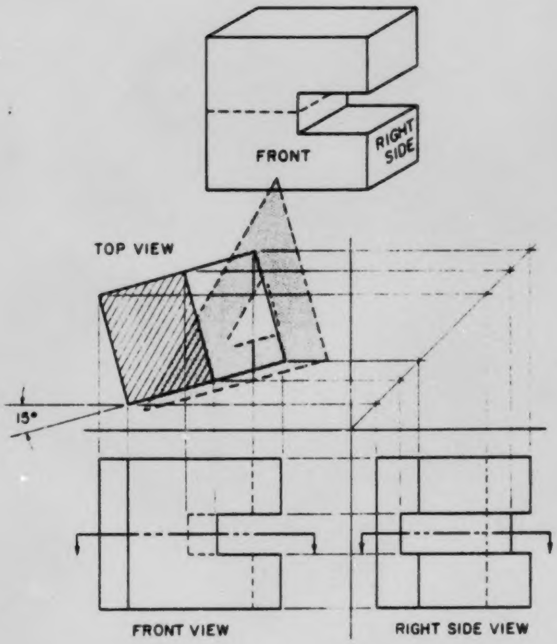
45.273
Figure 5-48.—Diagonal hatching on auxiliary section.

In the upper part of figure 5-51 there is a two-view multiview projection of a block. Though the line AB is parallel to the horizontal plane of projection, it is oblique to both the vertical and the profile planes. It is therefore a non-normal line, which will be a nonisometric line in an isometric projection or drawing of the same object. All of the other lines in the two-view multiview projection are normal lines, however, which will appear as isometric lines in an isometric drawing or projection.

You cannot, then, transfer AB directly from the two-view multiview projection to an isometric drawing. You can, however, transfer directly all the normal lines in the multiview projection, which will be isometric lines, appearing in their true lengths, in the isometric drawing. When you have done this, you will have constructed the entire isometric drawing exclusive of line AB and its counterpart on the bottom face of the block. The end points of AB and its counterpart will be located, and it will only be necessary to connect them with straight lines.

ANGLES IN ISOMETRIC

In an isometric drawing or projection an angle on the original object never appears in its true size in the drawing. Even the angle formed by two normal lines, such as one of the 90° corner angles on the block shown in figure 5-51, appears distorted in isometric.



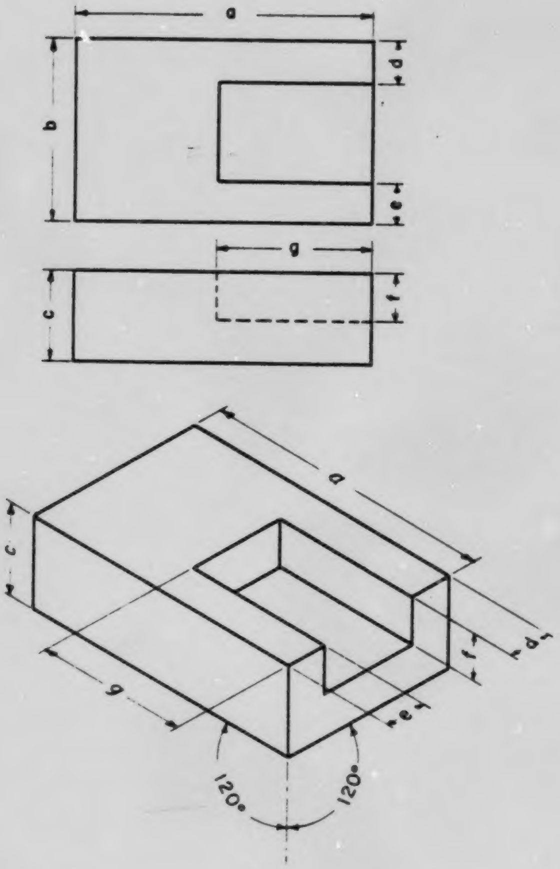
45.274
Figure 5-49.—Diagonal hatching on a revolution.

To transfer an angle like the one shown in figure 5-52 to an isometric drawing or projection you follow the same principle used in transferring a nonisometric line. In the block shown in figure 5-52, top view, line AB makes a 40° angle with the front edge. This line is a non-normal line which will appear as a nonisometric line in an isometric drawing. You locate the end points of AB on the isometric drawing by transferring distances a and b, measured along normal lines in the multiview projection and along corresponding isometric lines in the isometric projection.

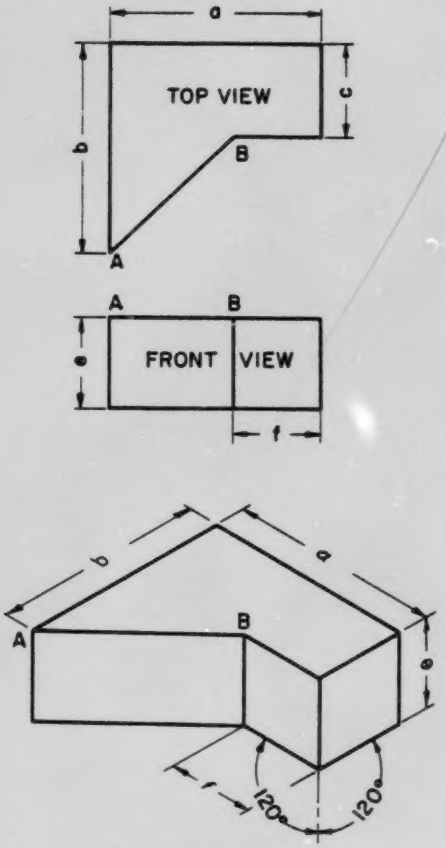
The angle which measures 40° on the multiview top view measures only about 32° on the isometric drawing.

CIRCLES IN ISOMETRIC

A circle which appears in a multiview orthographic drawing will appear as an ellipse on



45.275
Figure 5-50.—Box construction.



45.276
Figure 5-51.—The line AB is a nonisometric line.

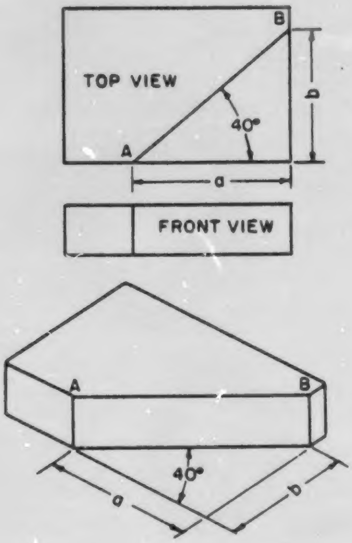
conjugate axes of equal length in an isometric drawing of the object. The legs of the conjugate axes are drawn parallel to the upper legs of the isometric axes. This is illustrated in figure 5-53. The circle which appears in the top view appears as an ellipse on conjugate axes of equal length in the isometric drawing. The conjugate axes of the ellipse are equal in length to the diameter of the circle shown in the top view, and are made parallel to the diagonal legs of the isometric axes.

You could draw the ellipse by the method previously described for drawing an accurate ellipse on conjugate diameters. However, an approximate ellipse, drawn with a compass, is usually satisfactory. The method of drawing an approximate ellipse on equal conjugate diameters is illustrated in figure 5-54. First draw the circumscribing parallelogram ABCD, by drawing at the end of each axis lines which are parallel to the other axis. Next construct the approximate ellipse as follows:

Draw the diagonal AC. From the midpoint of AB and the midpoint of BC draw lines to D. The points of intersection between these lines and the diagonal AC (points 0 and 0') are the centers for the end arcs of the approximate ellipse. Points B and D are the centers for the side arcs.

NONCIRCULAR CURVES IN ISOMETRIC

A line which appears as a noncircular curve in a regular multiview view of an object will



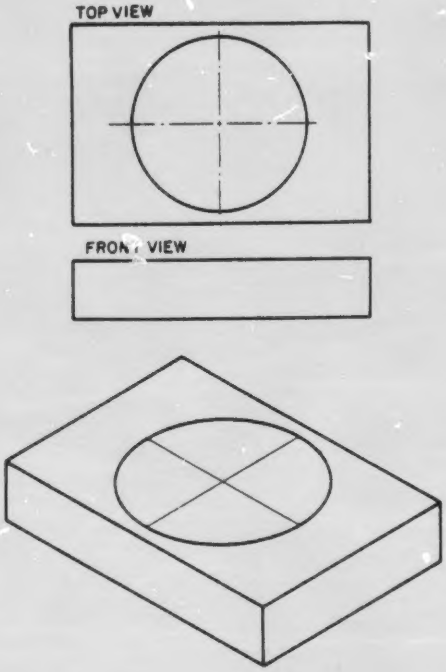
45.277
Figure 5-52.—Drawing an angle in isometric.

obviously appear as a nonisometric line in an isometric drawing or projection of the object. To transfer such a line to an isometric drawing or projection you must plot a series of points along the line, by measurements taken along normal lines in the multiview view and transferred to corresponding isometric lines in the isometric drawing or projection.

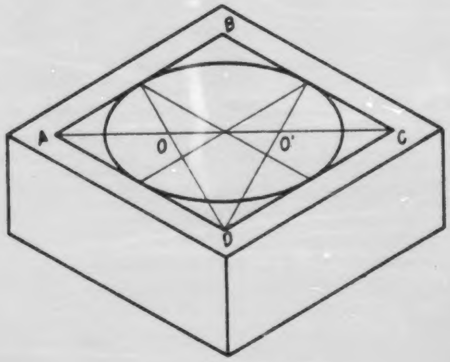
In the upper part of figure 5-55 there is a two-view multiview projection of a block with an elliptical edge. To make an isometric drawing of this block, first complete the circumscribing rectangle in the upper view, lay off equal intervals as shown, and drop perpendiculars from these intervals to the elliptical edge of the block. Next construct the block as a rectangle in isometric, as shown in the lower part of figure 5-55, and plot a series of points along the elliptical edge by laying off the same perpendiculars shown in the top multiview view. Fair in the line of the ellipse through these points with a french curve.

ALTERNATE POSITIONS OF ISOMETRIC AXIS

Up to now the isometric axis had always been shown with lower leg vertical. The axis may,



45.278
Figure 5-53.—Circle in orthographic appears as ellipse in isometric.



45,279
Figure 5-54.—Method of drawing approximate ellipse on equal conjugate axes.

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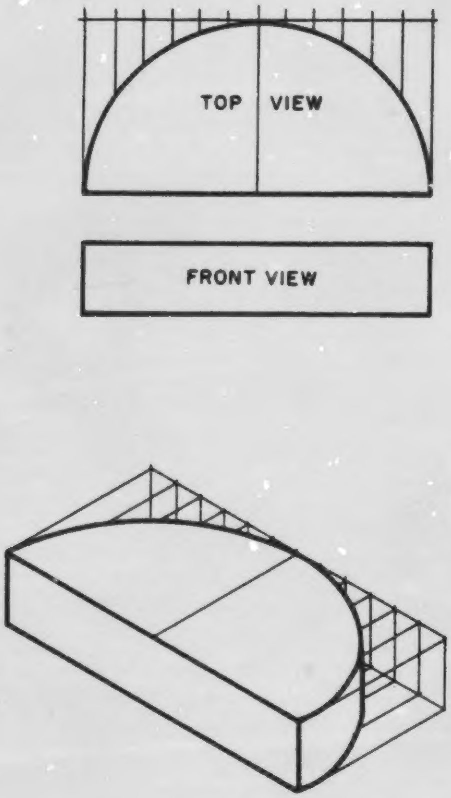


45.280
Figure 5-55.—Method of drawing a noncircular curve in isometric.

however, be used in any position, provided that the angle between adjacent legs is 120°. Figure 5-56 shows how the position of the isometric axis varies with different views of an object.

DIAGONAL HATCHING IN ISOMETRIC

Diagonal hatching on a sectional surface should appear to make a 45° angle with the horizontal or vertical axis of the surface. If the sectional surface is an isometric surface (one which makes an angle of 35° 16' with the plane of projection), lines drawn at 60° to the horizontal of the paper, as shown in figure 5-57, present the required appearance. For diagonal hatching on a nonisometric surface you must experiment to determine the angle which presents the required appearance.

OBLIQUE DRAWING TECHNIQUES

Earlier in this chapter, a comparison was made between an orthographic projection of a cube whose nearest face was parallel to the plane of projection and an oblique projection of the same cube in the same position. You have seen that the orthographic view shows only one surface, the front view, while the oblique view shows three surfaces—the front, the top, and the side. The projection lines of the orthographic view are parallel to each other and are perpendicular to the plane of projection. It is the same case with the oblique view, except that the projectors are not perpendicular to the plane of projection.

In an oblique projection of a rectangular object one face of the object (usually the most prominent or most important face) is placed parallel to the plane of projection. The resulting projection will show this face in its true dimensions, and with angles of the same sizes as on the original object. Angles on the other faces, however, will be distorted.

THE OBLIQUE AXES

As in isometric drawing, there are three lines in oblique axes, two of which are always perpendicular to each other, and assumed to be resting on the plane of the paper. The third line is a receding line that is drawn at any convenient angle with the horizontal. The choice of this convenient angle should be one that permits showing the details on the desired receding surface as clearly as possible. This is generally a matter of convenience to the draftsman. There is no fixed requirement in positioning the oblique axes on the drawing sheet, except that the two axes resting on, or parallel to, the plane of the paper must be 90° apart. The various positions in which an oblique axis may be drawn are shown in figure 5-58.

IRREGULAR LINES

A line which would be a normal line in a multiview projection or an isometric line in an isometric projection is called in an oblique projection a REGULAR line. It follows that in an oblique projection an IRREGULAR line is one which would be a non-normal line in a multiview projection and a nonisometric line in an isometric projection.

As an isometric line is parallel to one or another of the isometric axes, so is a regular line parallel to one or another of the oblique axes. However, all lines on the face which are parallel to the plane of projection are regular in an oblique projection. A regular line appears projected in its true dimensions.

In the upper part of figure 5-59 there is a two-view multiview projection of a block. The line AB is a nonnormal line, which appears in the oblique projection below as an irregular line. To transfer the line you proceed as you do with a nonisometric line. You draw the projection by measurements made along regular lines, and this locates the end points of the irregular line. The procedure for a cabinet projection would be the same, except that measurements made along the receding axis would be reduced by half.

ANGLES IN OBLIQUE

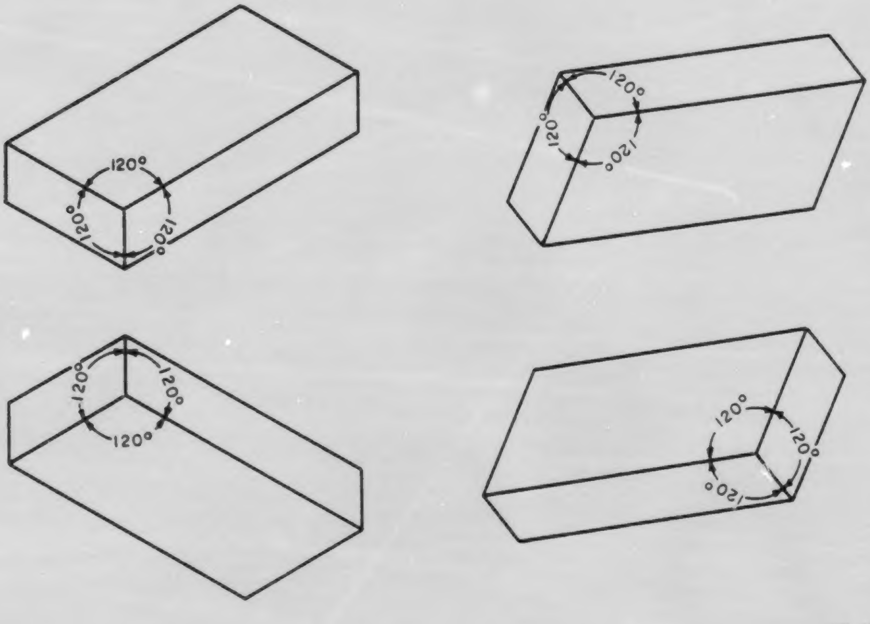
In an oblique projection an angle on the surface which is parallel to the plane of projection will appear in its true size; an angle on any other surface will not. In the upper view of figure 5-60 there is a two-view multiview projection of a block. There is a 30° angle drawn on the surface of the top view and another on the surface of the front view. The oblique projection below the angle on the front view (which is parallel to the plane of projection) still measures 30°. The angle on the top view, however, now measures only about 9°. You transfer the top view angle just as you do an angle in isometric, by locating the end points of the line by measurements made along normal lines.

CIRCLES IN OBLIQUE

In an oblique projection a circle drawn on the surface parallel to the plane of projection will project as a circle. A circle on any other surface will project as an ellipse on conjugate axes, each axis being parallel to a leg of the oblique axis.

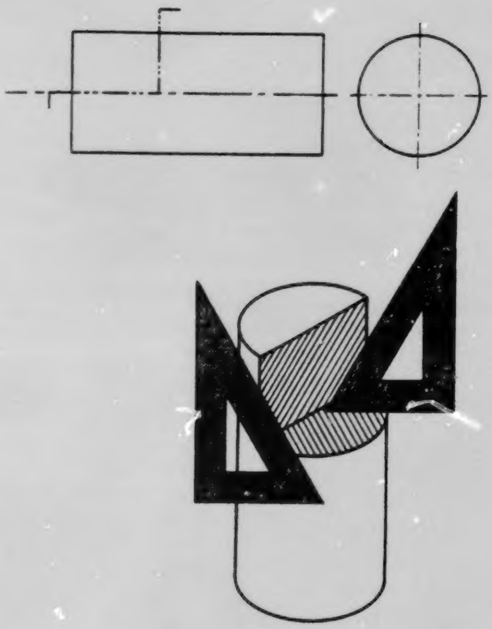
In the upper part of figure 5-61 there is a two-view multiview projection of a block with a circle drawn on its upper face. Below there is an oblique projection in which the circle appears as an ellipse. Each of the conjugate diameters of the ellipse is equal to the diameter of the circle.

Here again, you can draw an approximate ellipse with a compass, as shown in figure 5-62.

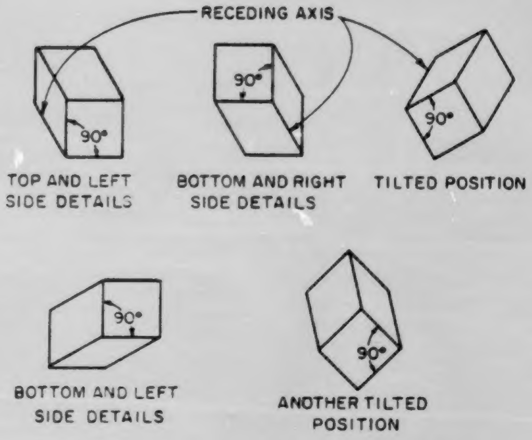


45.281

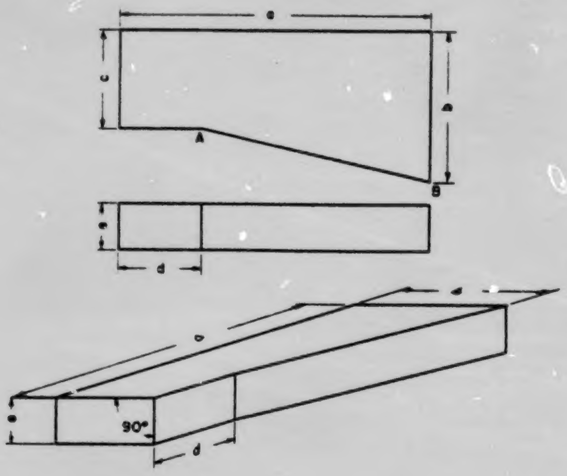
Figure 5-56.—Position of isometric axis varies with various views of object.



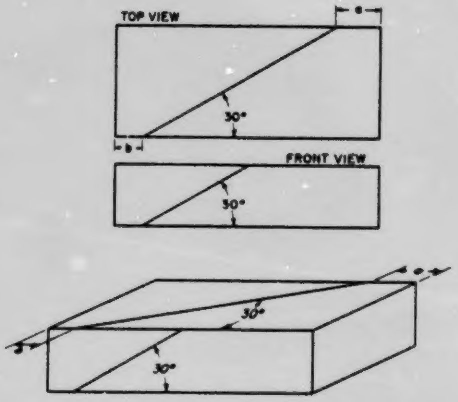
45.282
Figure 5-57.—Diagonal hatching in isometric.


RECEDING AXIS
90°
90°
90°
TOP AND LEFT SIDE DETAILS
BOTTOM AND RIGHT SIDE DETAILS
TILTED POSITION
90°
90°
BOTTOM AND LEFT SIDE DETAILS
ANOTHER TILTED POSITION

45.716
Figure 5-58.—Varying positions of the oblique axis to show desired details.



45.285
Figure 5-59.—Oblique projection of irregular line.



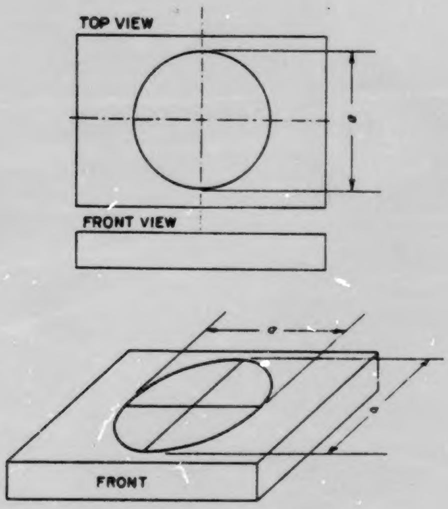
45.286
Figure 5-60.—Transferring an angle in oblique.

First draw your oblique axes lightly; then draw the circumscribing parallelogram ABCD, whose sides are equal to the diameter of the circle. These sides are drawn on the horizontal plane of the receding axis in this particular example. From the midpoints of each side, draw lines EF and GH. EF is parallel to BC, and GH is parallel to AB. Then draw perpendiculars from the midpoints of lines CD at F, and line AD at H, with the perpendiculars intersecting at X. The perpendiculars from the remaining sides intersect at X'. As you can see in figure 5-62, these perpendicular bisectors also intersect at points Y and Y'. Now with a compass and using these intersection points as centers, draw the corresponding arcs of the ellipse.

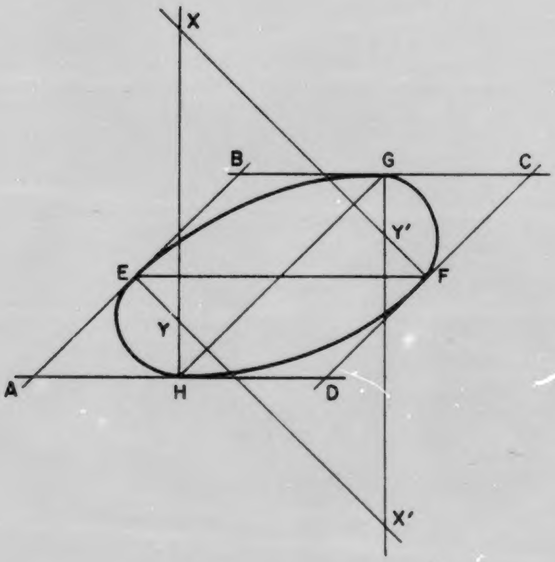
The method (called the four-center method) is basically the same as that shown in figure 5-54. It happens, however, that in an isometric projection the perpendicular bisectors from the sides intersect at corners of the circumscribing parallelogram. In an oblique projection this is not usually the case.

The situation with regard to circles is a little different in a cabinet projection, in which distances along the receding axis are reduced by half. In the upper part of figure 5-63 there is a two-view multiview projection of a block with a circle on the right side. Below there is an oblique projection in which the circle appears as an ellipse on conjugate axes. The axis which parallels a leg of the front axis has the same length as the diameter of the circle. The one which parallels the receding axis, however, has a length equal to only one-half the diameter of the circle when it is in a cabinet projection.

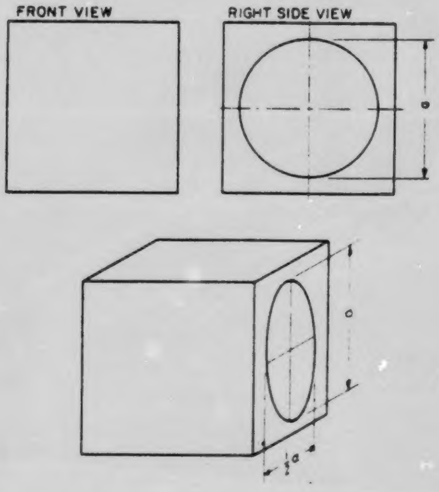
You must draw this ellipse by the method described in the previous chapter for drawing an ellipse on conjugate diameters. The four-center



45.287
Figure 5-61.—Oblique projection of a circle on a receding surface.



45.288
Figure 5-62.—Method of drawing approximate ellipse for oblique projection of a circle on a receding surface.



45.289
Figure 5-63.—Circle on receding surface shown in cabinet projection.

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CHAPTER 13

HORIZONTAL CONTROL

A system of control points must be established in order to definitely locate various points or details on the surface of the earth. Details must be located relative to some type of control system, either local or universal. In this way, the position of the detail will be established with some degree of permanency. Relative positions of detail points can be determined within a local control system, or if the control system is tied into geodetic control, the positions of all detail points are known with respect to a world wide system. The control starts with a triangulation network supplemented by traverse and forms the MAIN CONTROL SYSTEM. Stations of the main control scheme should be located close to the points to be tied in, to reduce auxiliary or supplementary control required. Supplementary control usually consists of one or more short traverses which are run close to or across a project area. Supplementary stations should be close together to afford easy tie-ins for the project. These stations should be established to the degree of accuracy required by the purpose of the survey.

A traverse which has been established and is used to locate detail points and objects is often spoken of as a CONTROL TRAVERSE. Any line from which points and objects are located is a CONTROL LINE. A survey is controlled horizontally by measuring horizontal distances and horizontal angles. This is often referred to as HORIZONTAL CONTROL.

If a main control system must be run in first, it should be of an accuracy to permit the supplementary survey to furnish the proper accuracy. For example, if third order accuracy is required for the detail, and supplementary control must be run for quite a distance, the main system must be of higher accuracy to maintain third order through the supplementary control. If the main system just meets third order requirements, any length of supplementary control will drop the accuracy below the limits and make it useless.

This chapter will discuss the operation of direction instruments such as the compass and the transit, various methods in determining the exact locations of detail points and objects, and describe various techniques employed by surveyors to bypass obstacles encountered during fieldwork.

DIRECTIONS AND DISTANCES

There are various ways of describing the horizontal location of a point, as was explained in chapter 8. In the last analysis, they are all reducible to the basic method of description, which is by stating the length and direction of a straight line between the point whose location is being described and a reference point.

Direction, like horizontal location itself, is also relative—that is, the direction of a line can only be stated relative to a "reference line" of known (or sometimes of assumed) direction. In true geographical direction the reference line is the meridian passing through the point where the observer is located, and the direction of a line passing through that point is described in terms of the horizontal angle between that line and the meridian. In magnetic geographical direction the reference line is the magnetic, rather than the true, meridian.

REFERENCE POINTS

The horizontal location of a point can only be stated in terms of the relative position of the point with reference to another point of known location. To say that the location of the other point is "known" means only that its relative position with regard to still another point of "known" location has been determined. Eventually the system of reference heads up in a point whose location is simply incapable of being described, because of the absence of any further reference points. In the geographical system of location by latitude and longitude, for instance, the ultimate reference point is a point where a meridian passing through the town of Greenwich, England, intersects the earth's Equator. The location of the Equator can be stated with reference to the earth's geographical poles; but the location of Greenwich, England, can only be stated as the location of Greenwich, England.

CONVERTING DIRECTIONS

The direction of a traverse line is commonly given by bearing. In field traversing, however, it is more convenient to turn deflection angles with a transit than it is to orient each traverse line to a meridian. The procedure for converting bearings to deflection angles is as follows.

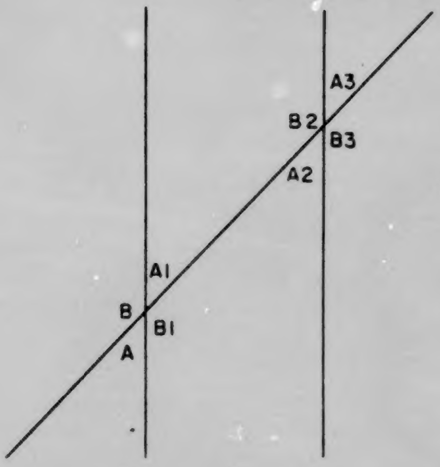
Converting Bearings to Deflection Angles

The conversion procedure is based on the well-known geometrical proposition illustrated in figure 13-1. This figure shows two parallel lines which are intersected by another line (called a "traverse"). It can be proved geometrically that the angles A and A_1, B and B_1, A_2 and A_3, and B_2 and B_3 are equal (vertically opposite angles). It can likewise be shown that angles $A = A_2$, $B = B_2$ (corresponding angles). So, angles $A = A_1 = A_2 = A_3$ and $B = B_1 = B_2 = B_3$. It can also be shown that the sum of the angles which together form a straight line is 180°, and the sum of all the angles around the point is 360°.

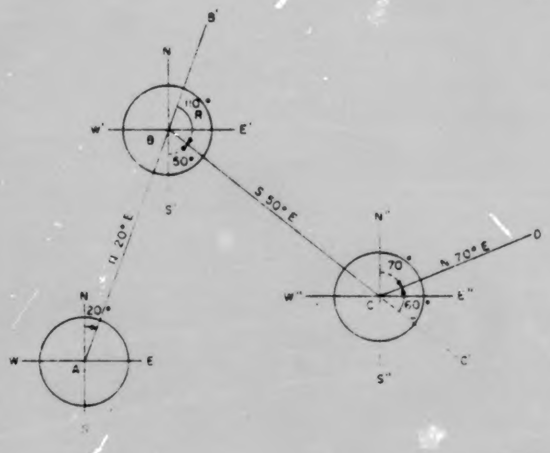
Figure 13-2 shows a traverse containing traverse lines AB, BC, and CD. The meridians through the traverse stations are indicated by the lines NS, N'S', and N''S''. Although meridians are not, in fact, exactly parallel, they are assumed to be so for conversion purposes. Consequently, we have here three parallel lines intersected by traverses, and the angles created will therefore be equal as illustrated in figure 13-1.

The bearing of AB is given as N 20° E, which means that angle NAB measures 20°. To determine the deflection angle between AB and BC you would proceed as follows. If angle NAB measures 20°, it follows that angle N'BB' must likewise measure 20°, because the two corresponding angles are equal. Now, the bearing of BC is given as S 50° E, whch means that angle S'BC measures 50°. The sum of angle N'BB' plus S'BC plus the deflection angle between AB and BC (angle B'BC) is 180°. Therefore, the size of the deflection angle is 180° - (N'BB' + S'BC), or 180° - (50° + 20°), or 110°. The figure indicates that the angle should be turned to the right; therefore, the complete deflection angle description is 110°R.

The bearing of CD is given as N 70° E; therefore, angle N''CD measures 70°. Angle S''CC' is equal to angle S'BC, and therefore



45.764
Figure 13-1. — Parallel lines (meridians) cut by a traverse.



45.765
Figure 13-2. — Converting bearings to deflection angles.

measures 50°. The deflection angle between BC and CD equals 180° - (S''CC' + N''CD), or 180° - (50° + 70°), or 60°. The figure indicates that the angle should be turned to the left.

Converting Deflection Angles to Bearings

Converting deflection angles to bearings is simply the same procedure applied for a different end result. Suppose that in figure 13-2 you know the deflection angles and want to determine the corresponding bearings. To do this you must know the bearing of at least one of the traverse lines. Let's assume that you know the bearing of AB and want to determine the bearing of BC. You know the size of the deflection angle B'BC is 110°. The size of angle N'BB' is the same as the size of NAB, which is 20°. The size of the angle of bearing of BC equals 180° - (B'BC + NAB), or 180° - (110° + 20°), or 50°. The figure shows you that BC lies in the second or SE quadrant; therefore, the full description of the bearing is S 50° E.

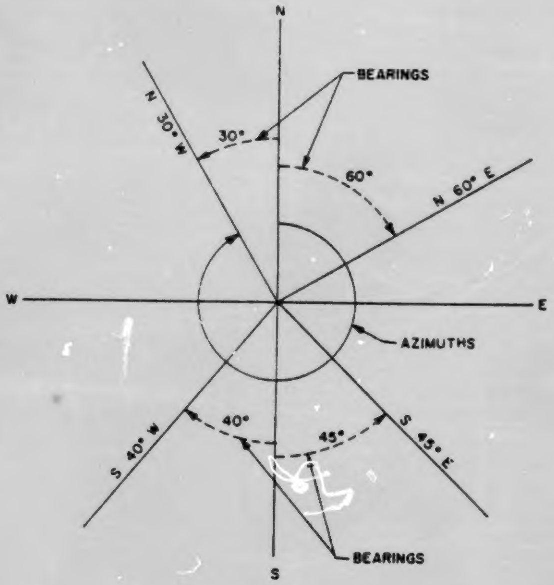
Converting Bearings to Interior/Exterior Angles

Converting a bearing to an interior or exterior angle is, once again, the same procedure applied for a different end result. Suppose that in figure 13-2 angle ABC is an interior angle and you want to determine the size. You know that angle ABS' equals angle NAB, and therefore measures 20°. You know from the bearing of BC that angle S'BC measures 50°. The interior angle ABC equals ABS' + S'BC, or 20° + 50°, or 70°.

The sum of the interior and exterior angles at any traverse station equals the sum of all the angles around a point, or 360°; this term is sometimes referred to as CLOSING THE HORIZON. Therefore, the exterior angle at station B equals 360° minus the interior angle, or 360° - 70°, or 290°.

Converting Azimuths to Bearings or Vice Versa

Suppose you want to convert an azimuth of 135° to the corresponding bearing. This azimuth is greater than 90°, but less than 180°; therefore, the line lies in the SE quadrant. As shown in the figure 13-3, the bearing angles are always measured from the north and south end of the reference meridian. (When solving any bearing problem, draw a sketch to get a clear picture.) As for the azimuth, the horizontal direction is reckoned clockwise from the meridian plane. It is measured between either the north end or the south end of the reference meridian and the line in question. When we talk about azimuth in this training manual, however it must be understood that it is referenced clockwise from the NORTH point of the meridian. The numerical value of this 135° azimuth angle is measured from N. The value of the bearing in this case, then, is 180° - 135°, or 45°. The complete description of the bearing, then, is S 45° E.



45.766
Figure 13-3.—Converting of azimuths to bearings, or vice versa.

For example, if you want to convert a bearing of N 30° W into an azimuth angle, you know that the angle location must be in the NW quadrant. Then, draw an angle of 30° from the north end of the reference meridian. Since we measure azimuth angles clockwise from the north end of the reference meridian. In this case, we subtract 30° from 360°, then we have 330°. Therefore, the bearing of N 30° W is equal to 330° azimuth angle.

DIRECTION BY MAGNETIC COMPASS

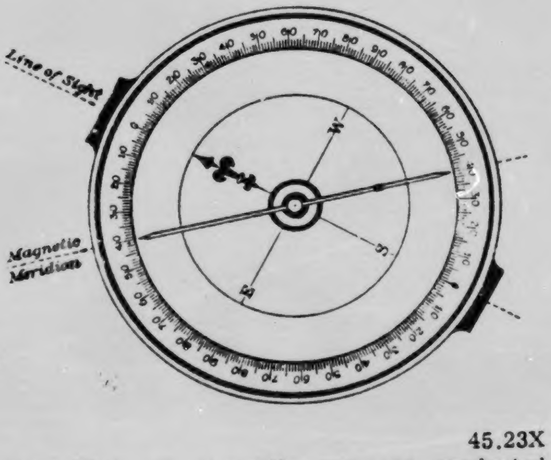
Figure 13-4 shows a magnetic compass graduated in quadrants for taking bearings. Note that on a compass of this type the E and W indications are in the opposite positions from those of. the E and W indications on a map or chart.

The observer is determining the magnetic bearing of the dotted line labelled "line of sight". He first mounted the compass on a steady support, leveled it, and waited for the needle to stop oscillating. He then carefully rotated the compass until the North-South line on the card lay exactly along the line whose bearing he was seeking.

The bearing is now indicated by the needle point. The needle point indicates a numerical value of 40°. The card indicates the NE quadrant. The magnetic bearing is therefore N 40° E.

Correcting for Local Attraction

Figure 13-4 represents the compass needle as lying along the "magnetic meridian". This in turn means either that the compass is in an area free of local attraction, or that the effect of local attraction has been eliminated by adjusting the compass card as described later. Local attraction means the deflection of the compass needle by a local magnetic force (such as that created by nearby electrical equipment or by a mass of metal such as a bulldozer). When local attraction exists and is not compensated for, the bearing you get is a "compass" bearing. A compass bearing does not become a "magnetic" bearing until it has been corrected for local attraction. Suppose, for example, that you read a compass bearing of N 37° E. Suppose that the effect of the magnetic attraction of a nearby (say) pole transformer is such as to deflect the compass needle 4° to the W of the magnetic meridian. This means that, in the absence of this local attraction, the compass would read, not N 37° E, but N 41° E. The correct magnetic bearing, then, is N 41° E.



45.23X

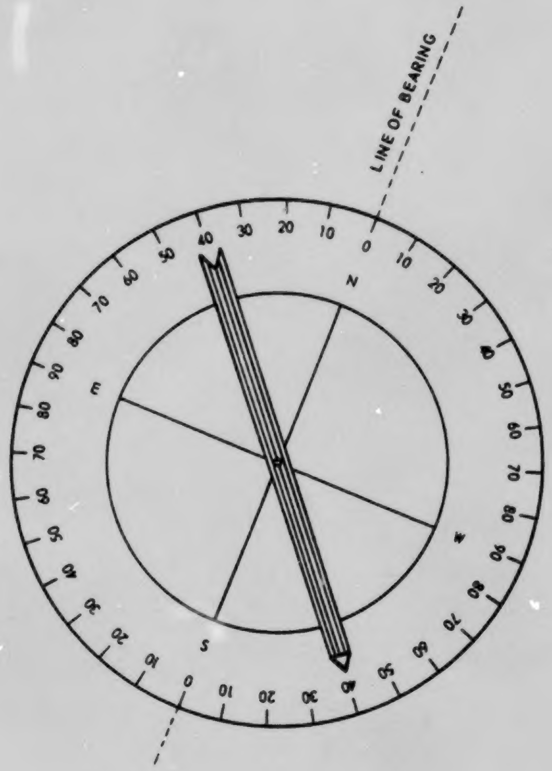
Figure 13-4.—A magnetic compass graduated for bearing observations.

To correct a compass bearing for local attraction, the first thing you must do is determine the amount and direction (E or W) of the local attraction. First set up the compass where you propose to take the bearing. Then select a distant object which may be presumed to be outside the range of any local attraction. Take the bearing of this object. Suppose you read a bearing of S 60° W.

Now shift the compass to the immediate vicinity of the object you sighted on, and take the bearing from there of the original setup point. In the absence of any local attraction at the original setup point, you would read the back bearing of the original bearing, or N 60° E. Suppose instead that you read N 48° E. The back bearing of this is S 48° W. Therefore, the bearing as indicated by the compass under local attraction is S 60° W, but as indicated by the compass not under attraction it is S 48° W. The amount and direction of local attraction are therefore 12° W.

The question of whether you add the local attraction to, or subtract it from, the compass bearing to get the magnetic bearing depends on (1) the direction of the local attraction, and (2) the quadrant the bearing is in. As we've seen, for a bearing in the NE quadrant you add a westerly attraction to the compass bearing to get the magnetic bearing. It follows that in this quadrant you would subtract an easterly attraction from the compass bearing to get the magnetic bearing.

But consider now the compass shown in figure 13-5. This compass indicates a bearing of S 40° W. Suppose the local attraction is 12° W. The needle, then, is 12° west of where it would be without local attraction. You can see that in this quadrant you would subtract westerly attraction, from which it follows that you would add easterly attraction.



45.665
Figure 13-5.—Bearing by magnetic compass.

If you study figures 13-4 and 13-5 further, you will see that the situation with regard to adding or subtracting corrections is the opposite in diagonally opposite quadrants—that is: in the NE quadrant you subtract easterly and add westerly attraction; but in the SW quadrant you add easterly and subtract westerly. Similarly, in the NW quadrant you add esterly and subtract westerly attraction; but in the SE quadrant you subtract easterly and add westerly.

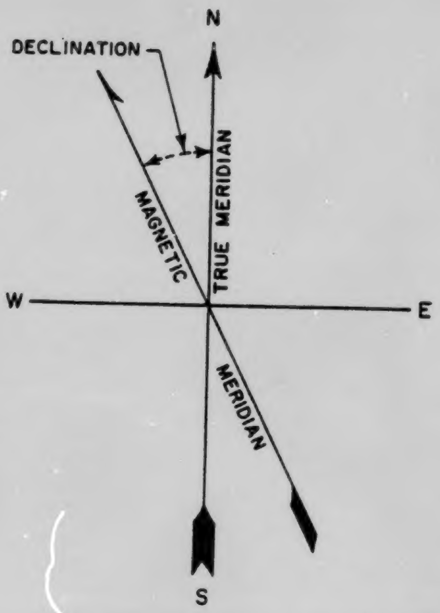
Magnetic Declination and Dip

The angle between the true meridian and the magnetic meridian is called MAGNETIC DECLINATION. Supposing the north end of the compass needle is pointing to the east of the true meridian, the declination is said to be east. If the declination of the north end of the compass needle is pointing to the west of the true meridian, the declination is said to be west (see figure 13-6).

The magnetic needle aligns itself with the earth's magnetic field and points toward the earth's magnetic pole. In horizontal projections these lines are inclined downward toward the north in the Northern Hemisphere, and downward toward the south in the Southern Hemisphere. Since the bar takes the position parallel with the lines of force, it becomes inclined with the horizontal. This phenomenon is called the MAGNETIC DIP.

Converting Magnetic Bearings to True Bearings

When you have corrected a compass bearing for local attraction, you have a magnetic bearing. As explained previously, in most areas of the earth a magnetic bearing differs from a true bearing by the amount of the local magnetic "declination" (called magnetic "variation" by navigators). The amount and direction of local declination are given on maps or charts of the area. Such a map or chart will usually state something like this: "Magnetic declination 26° 45' W (1950), annual increase 11'." This means



45.767
Figure 13-6.—Magnetic declination (west).

that if you are working in 1966 (16 years later), the local declination is 26° 45' + (11' x 16), or 26° 45' + 176', or 26° 45' + 2° 56', or 29° 41'.

To convert a magnetic bearing to a true bearing, you apply the declination to the magnetic bearing in precisely the same way that you apply local attraction to a compass bearing.

When you have a compass bearing and you know both the local attraction and the local declination, you can go from compass bearing to true bearing in a single process by applying the "algebraic sum" of local attraction and local declination. Suppose, for example, that local attraction was 6° W and declination 15° E. You could correct for local attraction and convert from magnetic to true in the same operation, by applying a correction of 9° E to the compass bearing.

Uncorrecting and Unconverting

You "correct" a compass bearing to a magnetic bearing by applying the local attraction, and you "convert" a magnetic bearing to a true bearing by applying the local declination.

Now, it could be the case that you might be given a magnetic bearing, and you might have to figure the corresponding compass bearing; or you might be given a true bearing, and you might have to figure the corresponding compass bearing on the basis of both local attraction and local declination.

The terms used to describe these procedures are, for the want of any better expressions, "uncorrecting" and "unconverting". All you need to remember is that when you are uncorrecting/unconverting you apply local attraction and local declination in the REVERSE of the directions in which you would apply them if you were correcting/converting.

Suppose, for example, that with a compass affected by a 10° W local attraction, you want to lay off a line bearing S 28° W magnetic by compass. If you were correcting, you would subtract a westerly attraction in the SW quadrant. However, for uncorrecting you ADD a westerly attraction in that quadrant. Therefore, to lay off a line bearing S 28° W, you would lay off S 38° W by the compass.

The same rule applies to azimuths. Suppose you have an azimuth-reading (measured from N) compass set up where local attraction is 10° W and declination is 25° E, and you want to lay off a line with true azimuth 256°. The algebraic sum of these is 15° E. For correcting/converting azimuths you ADD easterly and SUBTRACT westerly corrections; therefore, if you were correcting/converting you would add the 15° to 256°. Because you are uncorrecting/unconverting, however, you subtract; and to lay off a line with true azimuth 256° you read 241° by the compass.

Orienting a Compass

Some transit compasses, and practically all surveyor's and forester's "field" compasses are equipped for offsetting local attraction, local declination, or the algebraic sum of the two, by rotating the compass card as shown in figure 13-7. The upper view shows a compass bearing of N 40° W on a compass presumed to be affected by a local attraction of 10° W. In this quadrant you subtract westerly attraction; therefore, the magnetic bearing is N 30° W.

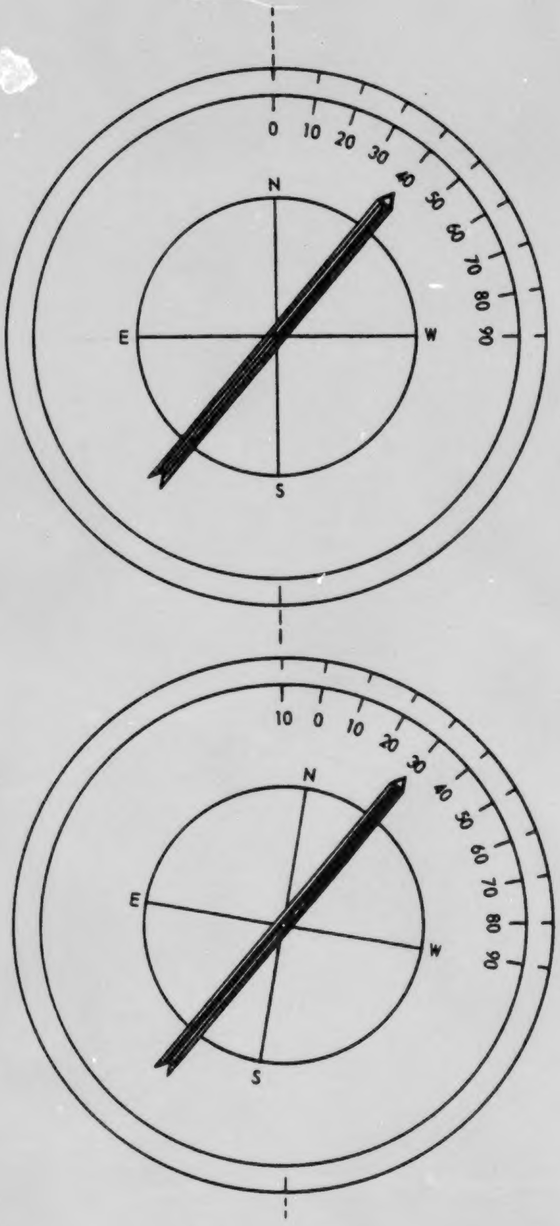
In the lower view the compass has been oriented for an error of 10° W, by simply rotating the compass card 10° W of its normal position. On most orienting compasses the card can be released for rotating by backing off a small screw on the face of the card. Note that you now read the correct magnetic bearing of N 30° W.

Compass-Tape Survey

Figure 13-8 shows field notes for a compass-tape survey of a small field. The compass used was a surveyor's compass. Although the compass was graduated only to the nearest 30', the instrumentman tried to estimate to the nearest one-third of the graduation—that is, to the nearest 10'—perhaps not a very realistic proposal. Local declination of 11° 32' W was "set off" by orienting the compass as previously described. Local attraction was either nonexistent, small enough to be ignored, or determined and also set off.

The compass was first set up at station A, shown in the sketch drawn on the "remarks" page. The first bearing taken was that of the line AE. This was actually the back bearing of EA, taken for the purpose of later checking against the forward bearing of EA.

Next the bearing of AB was taken, and the distance from A to B was chained. The bearing (S 62° 20' E) was entered beside B in the column headed "observed bearing". The chained distance was entered beside B in the column headed "distance".



45.667
Figure 13-7.—Orienting compass for 10° westerly correction.

The compass was shifted to station B, and the back bearing of AB (that is, the bearing of BA) was taken, as a check on the previously taken bearing of AB. The back bearing turned out to have, as it should have, the same numerical value (62° 20') as the forward bearing. A difference in the two would indicate either an inaccuracy in reading one bearing or the other, or a difference in the strength of local attraction.

Proceeding in this fashion, the party took bearings and back bearings, and chained distances, all the way around to the starting point at station A. The last forward bearing taken, that of EA, has the same numerical value as the back bearing of EA (bearing of AE) taken at the start.

Checking Accuracy of Observed Bearings

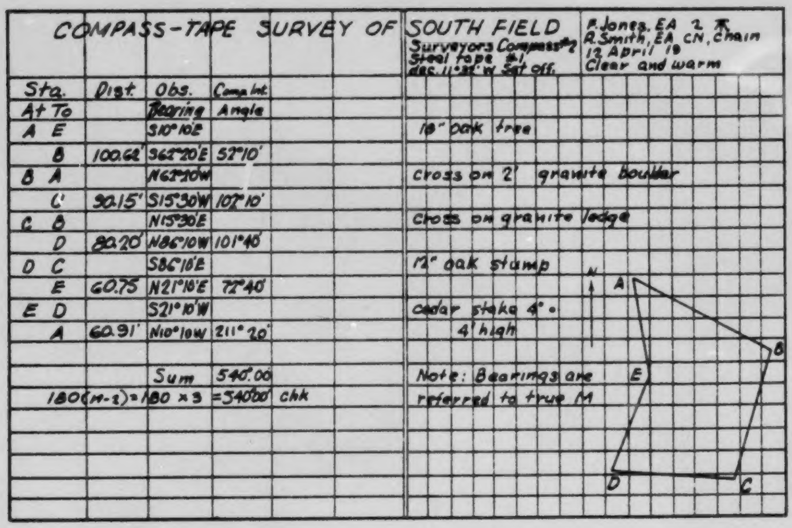
As a check on the accuracy of the whole bearing-reading procedure, the size of the interior angle at each station was computed from the observed bearings, by the procedure previously described for converting bearings to interior angles. The sizes of these angles were entered in the column headed "computed interior angle", and the sum was entered below.

The sum of the interior angles in a closed traverse should equal the product of 180° (n - 2), n being the number of traverse lines in the traverse. In this case the traverse has 5 lines; therefore, the sum of the interior angles should equal the product of 180° (5 - 2), or 180° x 3, or 540°. The computed sum is therefore the same as the added sum of the angles converted from observed bearings.

Compass Errors

If a magnetic compass has a bent needle, there will be a constant instrumental error in all observed bearings/azimuths. To check for this condition, set up and level the compass, wait for the needle to cease oscillating, and read the graduation indicated at each end of the needle. If the compass is graduated for bearings, the numerical value at each end of the needle should be the same. If the compass is graduated for azimuths, the readings should be 180° apart.

Similarly, if the pivot supporting the needle on a magnetic compass is bent, there will be an instrumental error in the compass. However, this error, instead of being the same for any reading, will be variable.



COMPASS-TAPE SURVEY OF SOUTH FIELD
Surveyors Compass #2
Steel tape #1
dec. 11°35' W Set off.

F. Jones, EA 2 ⊼
R. Smith, EA CN, Chain
12 April 19
Clear and warm

Sta. At	Sta. To	Dist.	Obs. Bearing	Comp. Int. Angle	
A	E		S10°10'E		18" oak tree
	B	100.62'	S62°20'E	52°10'	
B	A		N62°20'W		cross on 2' granite boulder
	C	90.15'	S15°30'W	107°10'	
C	B		N15°30'E		cross on granite ledge
	D	80.20'	N86°10'W	101°40'	
D	C		S86°10'E		12" oak stump
	E	60.75	N21°10'E	72°40'	
E	D		S21°10'W		cedar stake 4" □ 4' high
	A	60.91'	N10°10'W	211°20'	
			Sum	540°00	Note: Bearings are
180(n-2) = 180 × 3				= 540°00 chk	referred to true M

45.7.6

Figure 13-8.—Field Notes for compass-tape survey.

You can eliminate either of these instrumental errors by reading both ends of the needle and using the average between them. Suppose, for example, that with a compass graduated for bearings you read a bearing of N 45° E and a back bearing of S 44° W. You would use the average, or 1/2(45° + 44°), or N 44° 30' E.

The error in the compass should, of course, be corrected as soon as possible. Normally this is a job for an expert. With regard to the cause of a discrepancy in the reading at both ends, when such a discrepancy exists, it is more probable that the needle is bent than it is that the pivot is bent. If, after a needle discovered to be bent has been straightened, a discrepancy still exists, then it is probable that the pivot is bent as well.

If a compass needle is sluggish—that is, if it moves unusually slowly in seeking magnetic N—it will probably come to rest a little off the magnetic meridian. The most common cause of sluggishness is weakening of the magnetism of the needle. A needle may be remagnetized by drawing it over a bar magnet. The needle should be drawn from the center of the bar magnet toward the end, with the S end of the needle drawn over the N end of the magnet and vice-versa. On each return stroke the needle should be lifted well clear of the magnet.

Sometimes the cause of a sluggish needle is bluntness of the point of the pivot. This may be corrected by sharpening the pivot with a very fine file.

If the compass is not level when a bearing/azimuth is read, the reading will be in error. A similar error will exist if the compass is equipped with sighting vanes and the vanes, or one of them, is bent. To check for this last condition, set up and level the compass and sight with the vanes on a plumb bob cord.

The most common personal error in compass work is misreading caused by the fact that the observer's eye at the time of reading is not vertically above the needle point. Common mistakes are reading a needle at the wrong end and setting off local attraction and/or declination in the wrong direction when orienting a compass.

DIRECTION BY TRANSIT

In determining direction by transit you again measure the size of the horizontal angle between the line whose direction is sought and a reference line. With a transit, however, you are able to do this with considerably more accuracy and precision. Before proceeding to discuss the actual measurement of directions with a transit however, it is best that you know the proper procedure in setting up the transit.

Setting Up the Transit

The point at which the line of sight, the horizontal axis, and the vertical axis of a transit meet is called the instrument center. A point on the ground over which the center of the instrument is placed is the INSTRUMENT POINT, TRANSIT POINT, or STATION. The transit point or station usually is marked with a tack in a stake. Then the transit must be set over this definite point. Do not select a place for the transit station where the transit may be jarred or displaced. Avoid stations on loose planking, soft or marshy ground, or other places where the legs of the tripod may be moved easily.

When you are ready to use the transit, center the instrument as closely as possible over a definite point. For the purpose of centering the instrument over a point, a plumb line is suspended from a hook and chain beneath the instrument. The plumb string is tied with a slip knot, or some other device is used, so that the height of the plumb can be adjusted. Move the tripod legs as necessary until the plumb bob is about 1/4 in. of being over the tack, and making sure that the foot plate is nearly level. The tripod legs are spread and sufficient pressure is then applied to the legs, to ensure firmness in the ground and at the same time observing that the plumb bob did not move farther from the tack or center point mark. It is preferable to have a pair of opposite leveling screws in line with the approximate line of sight. Now, turn the plates so that each plate level is parallel to a pair of opposite leveling screws. The leveling screws are tightened in such a way that they are firm, but not tight. Rotate opposite pairs of leveling screws either toward each other or away from each other until the plate bubbles are centered.

If the plumb bob is not exactly over the point, loosen two adjacent leveling screws enough to free the shifting plate. Now slide the instrument head until the plumb bob is directly over the center of the tack.

If the plate bubbles are now somewhat off center, re-level the instrument. Then check the plumb bob position again. If the wind is blowing, stand on the windward side to shield the plumb line and bob from the breeze. Sometimes in very windy locations it may be helpful to construct a wind shield.

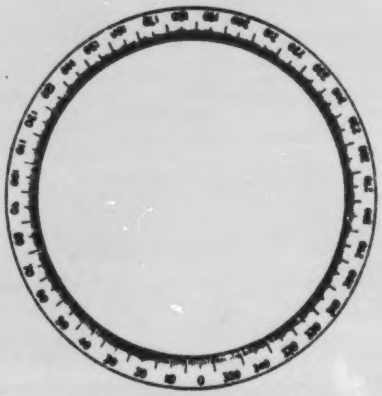
You must remember the following points in the operation of a transit:

1. The plate bubble follows the direction of the left thumb when manipulating the leveling screws.
2. Always check to see if the plumb bob is still over the point after leveling. If the plumb bob has shifted, re-center the instrument.
3. When you loosen two adjacent leveling screws, you can shift the transit head laterally.
4. Always maintain contact between the leveling screw shoes and the foot plate.
5. Test the telescope for parallax before observations are begun.
6. Do not disturb the setup of the instrument until you are certain that all observations at that point are completed and roughly checked. Instrument can be moved from that setup only after checking with the party chief.
7. Before the transit is moved or taken up, center the instrument on the foot plate, equalize roughly the height of the leveling screws, clamp the upper motion (the lower motion may be clamped lightly), and point the telescope vertically upward and also clamp lightly.

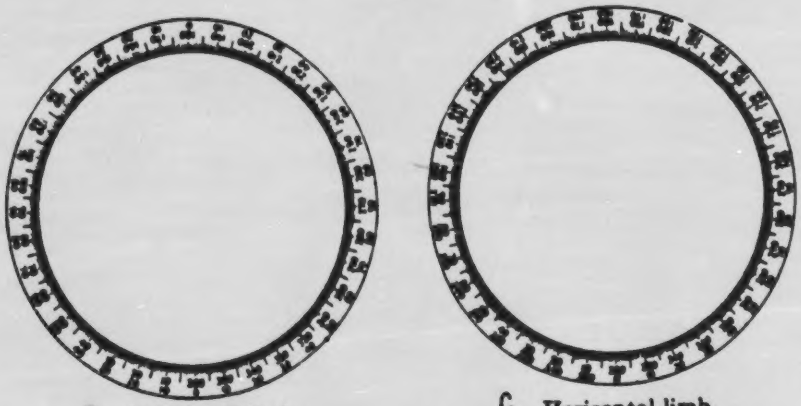
The setting and leveling of the transit expeditiously requires on your part a skill that is only attained through constant practice. During your apprenticeship, when you don't have the opportunity of operating the instruments during fieldwork, you can practice setting and leveling the instruments during break periods. Your senior EA however, has the responsibility of seeing to it that a certain period during the week be set aside for training in order to increase your skill in handling surveying instruments. If this is not feasible due to work pressure, then take advantage of any opportunity that you can to operate the instruments. Perhaps you can be the instrumentman on surveys that do not require a high degree of accuracy.

Measuring Horizontal Angles

The transit contains a graduated horizontal circle called the "horizontal limb". The horizontal limb may be graduated clockwise from 0° through 360° as shown in figure 13-9(A); or clockwise from 0° through 360° and also in quadrants, as shown in figure 13-9(B); or both



A Horizontal limb, numbered 0-360.



B Horizontal limb, numbered 0-360. and in quadrants.

C Horizontal limb, numbered 0-360 and 360-0.

45.29

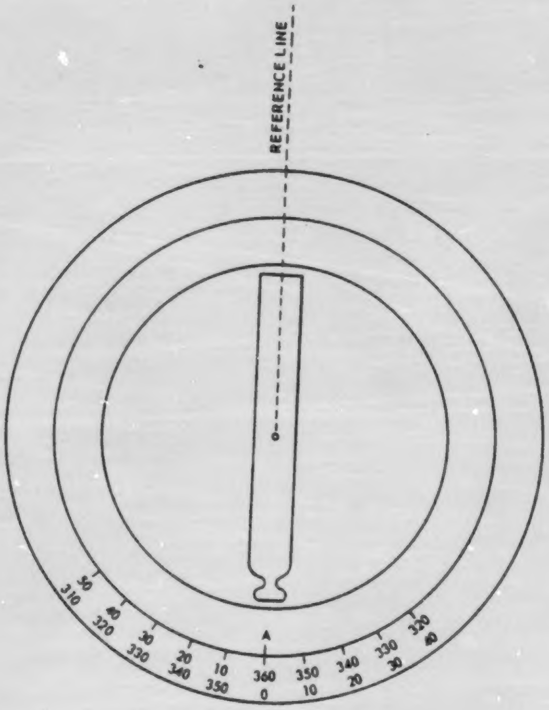
Figure 13-9.—Three ways of graduating horizontal limbs.

clockwise and counterclockwise from 0° through 360° as shown in figure 13-9(C).

The horizontal limb can be clamped to stay fast when the telescope is rotated (called "clamping the lower motion"), or it can be released for rotating by hand (called "releasing the lower motion").

The horizontal limb is paired with another circle called the "vernier plate", which is graduated only partially on either side of zero graduations located 180° apart on the plate. When the telescope is in normal (up-right) position, "A-vernier" is located vertically below the eyepiece and the "B-vernier" below the objective end of the telescope. The zero on each vernier is the indicator for reading on the horizontal limb the sizes of horizontal angles turned.

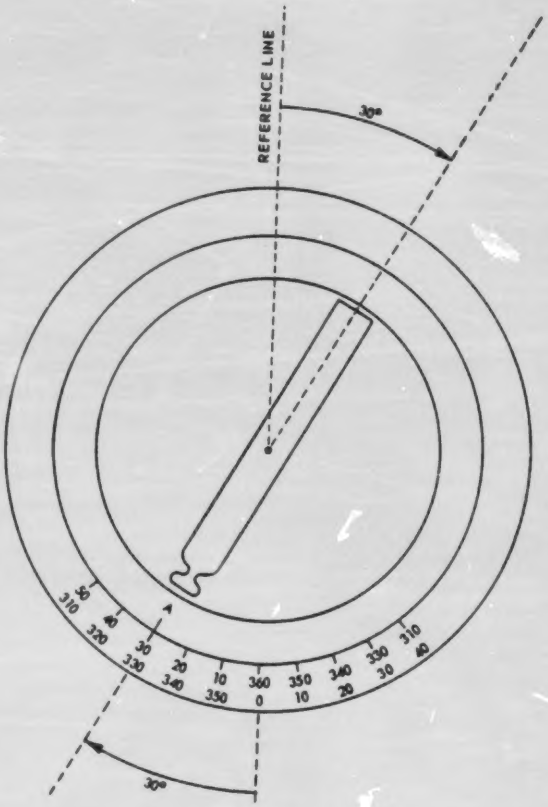
Figures 13-10 and 13-11 illustrate the method of "turning" an angle of 30° from a reference line with a transit. First, of course, you must plumb the instrument over the apex of the angle and carefully level it. Then bring one of the horizontal verniers near zero by hand; clamp the upper motion; and, by turning the upper tangent-screw, set one vernier at 0°—usually



45.668
Figure 13-10.—Setting the vernier at zero-zero.

you start with the A-vernier (see fig. 13-10). Sight the telescope approximately to the marker (range pole, chaining pin, and the like) held on the reference line; clamp the lower motion; and, by using the lower tangent-screw, set the line of sight exactly on the marker.

The upper motion is then released, and the telescope is rotated to bring the zero on the A-vernier in line with the 30° graduation on the horizontal limb, as shown in figure 13-11. To set the vernier exactly at 30°, again use the upper tangent-screw. Use a magnifying glass to set the vernier easily and accurately. Then mark the second point accordingly. You use the same procedure in measuring a horizontal angle except that instead of setting the angle, obviously, you sight on the existing points and take the angle reading after the two sightings. The vernier that was initially set at 0° gives the value of the angle (see chapter 9).



45.669
Figure 13-11.—Setting an angle exactly on the vernier zero.

The following suggestions may help when applied to horizontal angle measurement.

1. Make the centering of the line of sight as close as possible by hand so that you will not turn the tangent-screw through more than one or two revolutions. Make the last turn of the tangent-screw clockwise in order to compress the opposing springs.
2. Read the vernier with the eye directly over the top of the coinciding graduations to eliminate the effects of parallax.
3. As a check, take the reading of the other vernier. The readings should be 180° apart.
4. Check the plate bubbles before measuring an angle if they are centered, but do not disturb the leveling screws between the initial and final settings of the line of sight. If measuring angle

300

by repetition, the plate may be releveled after each reading, that is, before sighting again on the starting point.

5. When sighting at a range pole, the bottom of which is not visible, make sure that the flagman is holding it truly vertical, or let him use a plumb bob instead.

6. Avoid accidental movement of the horizontal circle, i.e., moving the wrong clamp or tangent-screw. If a number of angles will be observed from one setup without moving the horizontal circle, sight at some clearly defined distant object that will serve as a reference mark and take note of the angle. Occasionally, check this reading to this point during measurement to see if there is any accidental movement.

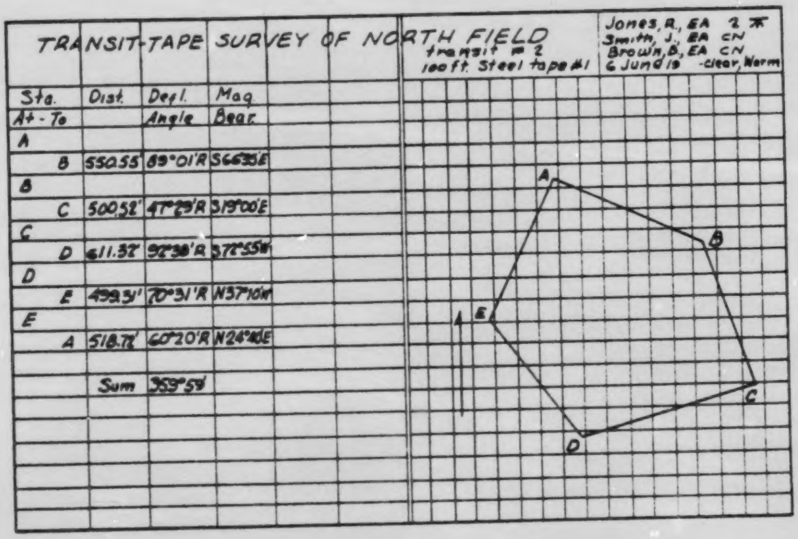
Transit Field Notes for Horizontal Angles

Figure 13-12 shows field notes for deflection angles, magnetic bearings, and distances for the closed traverse shown in the sketch. The transit was first set up at station A, and the magnetic bearing of AB was read on the compass. Then the deflection angle between the extension of EA and AB was turned, probably about as follows: The transitman released both clamps. He matched the vernier to zero by hand; tightened the upper motion clamp and set the zero exactly by using the upper tangent-screw. Then he plunged the telescope (inverted position) and trained the telescope by hand on a range pole held on station E. He then tightened the lower motion clamp and brought the vertical crosshair to exact alignment with the range pole by manipulating the lower motion tangent-screw.

With both clamps tightened, he replunged the telescope, which was then trained on the extension of EA. (Notice that the telescope is now in its normal position at this juncture.)

The transitman then released the upper motion and rotated the telescope to the right until the vertical crosshair came into line with a range pole held on station B. He set the upper motion clamp screw, and brought the vertical crosshair into exact alignment with the range pole by manipulating the upper motion tangent screw. He then read the size of the deflection angle (89° 01') on the A-vernier. Because the angle was turned to the right, he recorded "89° 01' R" in the column headed "deflection angle". The distance from A to B (550.55 ft) was chained and recorded in the column headed "distance".

The same procedure was carried out at each traverse station in a clockwise direction around



TRANSIT-TAPE SURVEY OF NORTH FIELD
transit # 2
100 ft. Steel tape #1

Jones, R., EA 2 ⊼
Smith, J., EA CN
Brown, B., EA CN
6 June 19 -clear, Warm

Sta. At - To	Dist.	Defl. Angle	Mag. Bear.
A			
B	550.55	89°01'R	S66°35'E
B			
C	500.52'	47°29'R	S19°00'E
C			
D	611.32'	92°38'R	S72°55'W
D			
E	499.31'	70°31'R	N37°10'W
E			
A	518.72'	60°20'R	N24°40'E
Sum	359°59'		

45.7.7

Figure 13-12. — Field notes for deflection-angle transit-tape survey.

the traverse to station E. Note the algebraic sum of the measured deflection angle (angles to the right considered as plus, to the left as minus), which is 359° 59'. For a closed traverse the algebraic sum of the deflection angles, from the standpoint of pure geometry, is 360° 00'. Therefore there is an "angular error of closure" here of 0° 01'. This small error would probably be considered normal error. A large error would indicate a mistake made somewhere along the line.

Closing the Horizon

In the preceding example the general accuracy of all the angular measurements was checked by comparing the sum of the deflection angles with the theoretical sum. The accuracy of a single angular measurement can be checked by the procedure called "closing the horizon". The method is based on the fact that the theoretical sum of all the angles around a point is 360° 00'.

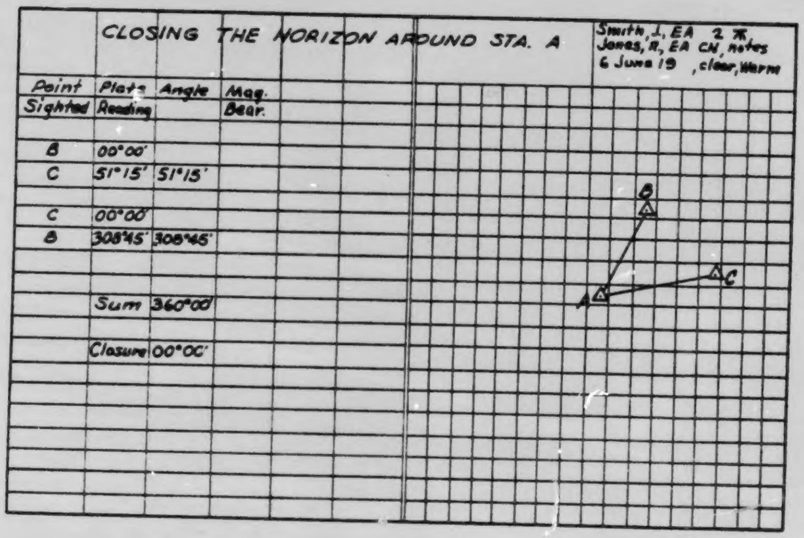
The field notes shown in figure 13-13 illustrate the procedure for closing the horizon. The transit was set up at station A and the angle BAC was turned, measuring 51° 15'. Then the angle from AC clockwise around to AB was turned, measuring 308° 45'. The sum of the two angles was 360° 00'. The angular error of closure was therefore 0° 00'—meaning that "perfect closure" was obtained.

Measuring Vertical Angles

Vertical circle and the vertical vernier of a transit were discussed in chapter 9 of this training manual. They are used for measuring vertical angles.

A vertical angle is the angle measured vertically up or down from a horizontal plane of reference. See view A of figure 13-14. When the telescope is pointed in the horizontal plane (level), the value of the vertical angle is zero. When the telescope is pointed up at a higher feature (elevated), the vertical angle increases from zero and is called a PLUS VERTICAL ANGLE or ANGLE OF ELEVATION. These values increase from 0° to +90° when the telescope is pointed straight up.

As the telescope is depressed (pointed down), the angle also increases in numerical value.



CLOSING THE HORIZON AROUND STA. A

Smith, J., EA 2 ⊼
Jones, R., EA CN, notes
6 June 19 , clear, Warm

Point Sighted	Plate Reading	Angle	Mag. Bear.
B	00°00'		
C	51°15'	51°15'	
C	00°00'		
B	308°45'	308°45'	
	Sum	360°00'	
	Closure	00°00'	

45.7.8
Figure 13-13.—Closing the horizon.

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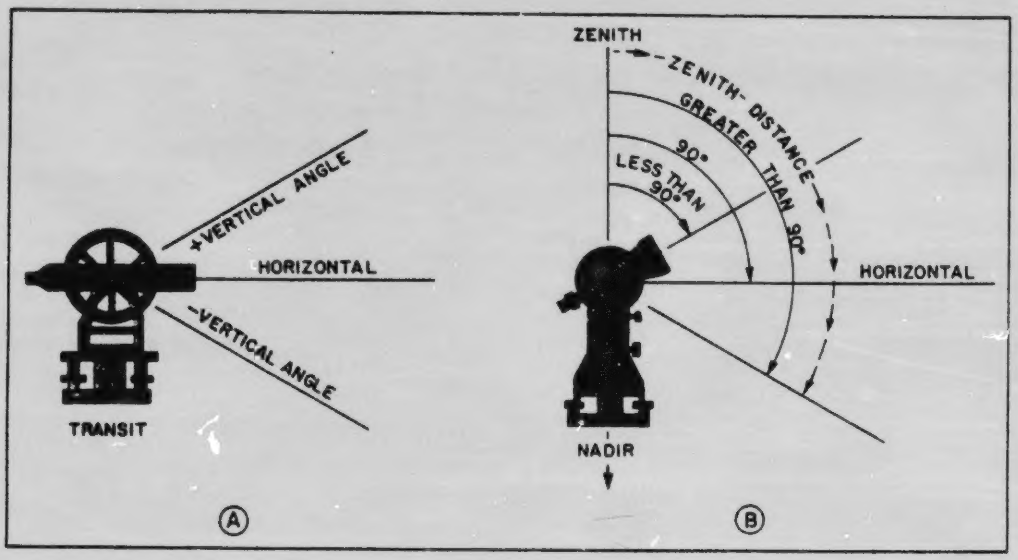


45.768
Figure 13-14. —Vertical angles and zenith distances.

A depressed telescope reading showing that it is below the horizontal plane is called a MINUS VERTICAL ANGLE or ANGLE OF DEPRESSION. These numerical values increase from 0° to -90° when the telescope is pointed straight down.

As in measuring horizontal angles, the transit must be set up on a definite point and leveled. The plate bubbles must be centered carefully—especially for transits that have fixed vertical vernier. The line of sight is brought approximately at the point, the horizontal axis is clamped, and then the horizontal crosshair is brought exactly at the point by means of the telescope tangent screw. The angle is read on the vertical limb by means of the vertical vernier.

On a transit with a movable vertical vernier, its vernier is equipped with a control level. The telescope is centered on the point as described above, but the vernier bubble is centered before the angle is read.

You will also measure vertical angles in trigonometric leveling, in astronomical observations, and the like. Trigonometric leveling will be discussed in the next chapter. In astronomical observations, you will meet the term ZENITH DISTANCE. It is used in solving astronomic triangles (see EA 1 & C) and a zenith distance is a vertical angle.

Zenith Distance

The ZENITH is an imaginary point overhead where an extension of the plumb line will intersect an assumed sphere on which the stars appear projected. The equivalent point which is directly below the zenith is called the NADIR. Use of the zenith permits reading angles in a vertical plane without using a plus or minus. Theodolites (chapter 9) have a vertical scale reading zero when the telescope is pointed at the zenith, instead of in a horizontal plane. With the telescope in a direct position and pointed straight up, the reading is 0°; on a horizontal line, the reading is 90°; and straight down, 180°. Vertical angle measurements with the theodolites (view B, figure 13-14), read angle of elevation as values less than 90°, and angle of depression as greater than 90°. These angle measurements with the zenith as the zero value are called the ZENITH DISTANCES. DOUBLE ZENITH DISTANCES are observations made with the telescope direct and reversed to eliminate errors due to the inclination of the vertical axis and the collimation of the vertical circle.

Measuring Angles by Repetition

You may recall from a previous discussion of the distinction between "precision" and "accuracy". A transit on which angles can be measured to the nearest 20 seconds is more precise than one on which they can be measured only to the nearest 1 minute, although it is not necessarily more accurate.

The inherent angular precision of a transit can be increased by the procedure called "repetition". To illustrate the principle of this procedure, suppose that with a 1 minute transit you turn the angle between two lines in the field and read 45° 00'. The inherent error in the transit is 1'; therefore, the true size of this angle is somewhere between 44° 59' 30" and 45° 00' 30".

Suppose, now, that you "repeat" this angle as follows. You leave the upper motion locked, but release the lower motion. The horizontal limb will now rotate with the telescope, holding the reading of 45° 00'. You plunge the telescope, train again on the initial line of the angle, and again turn the angle. You have now "doubled" the angle, and you should read approximately 90° 00' on the A-vernier.

Now, for this second reading the inherent error in the transit is still 1 minute, but the angle indicated on the A-vernier is about twice the size of the actual angle measured. The effect of this is to halve the total possible error. This error was originally plus or minus 30 seconds. It is now plus or minus only 15 seconds.

If you measure this angle a total of (say) 6 times, the total possible error will be reduced to one-sixth of 30 seconds, or plus or minus 5 seconds. In theory you could go on repeating the angle and increasing the precision indefinitely. In practice, however, because of lost motion in the instrument and accidental errors, nothing is gained by repeats in excess of 6 or 8.

The observation may be taken alternately with the telescope plunged prior to each subsequent observation, but it is much simpler if you take the first half of the observations with the telescope in the normal position, and the other half in inverted position. In the example given above, the first three readings may be taken when the telescope is in its normal position and the last three in the reversed position. To avoid the effect of tripod twist, after each repetition the instrument should be rotated on its lower motion in the same direction that it was turned during the measurement; that is, the direction of movement should be either always clockwise or always counterclockwise.

Measuring angle by repetition eliminates certain possible instrumental errors, such as those due to eccentricity and those caused by nonadjustment of the horizontal axis.

Figure 13-15 shows field notes for 6-time angles around a station. The angle BAC was measured 6 times, and the angle closing the horizon around station A was likewise measured 6 times. Grammatically speaking, of course, the first measurement is not a "repeat", but it is counted as such in the column headed "number of repetitions".

With the transit first trained on B and the zeros matched, the plate reading was 00° 00'. You see this entered beside B in the column headed "plate reading". For the first or "1-time" measurement of angle BAC the plate reading was 82° 45'. After 5 repeats the plate reading was 136° 28'.

Now, the sum of 6 angles, each measuring 82° 45', comes to 82° 45' x 6, or 496° 30'. Therefore, the plate reading of 136° 28' actually indicates a sum of 360° + 136° 28', or 496° 28'. The mean angle, then, is found by dividing 496° 28' by 6, which comes to 82° 44' 40". You see this entered under "mean angle".

The same procedure was followed with the closing angle, resulting in a mean angle of 277° 15' 20". The sum of the mean angle and mean closing angle is 360° 00'.

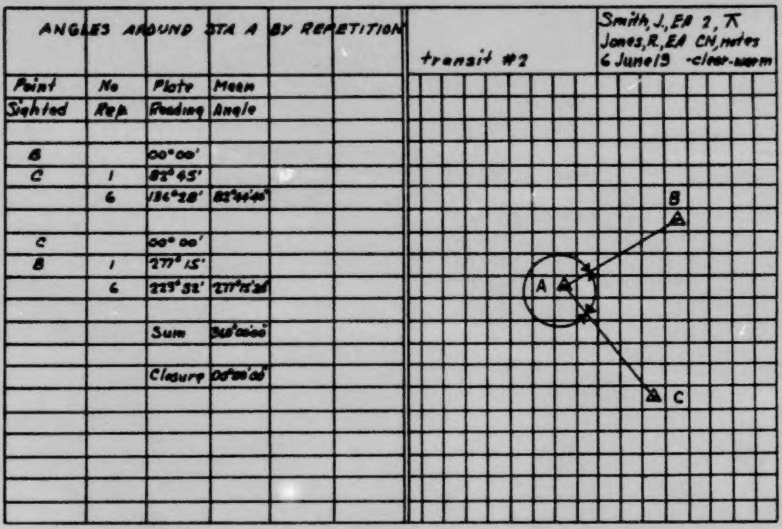
In actual practice, perfect angle closure would be unlikely. It has been assumed in these examples, simply to avoid complicating the discussion.

PROCEDURES IN RUNNING LINE

It is often necessary to extend a straight line marked by two points on the ground. One of the methods discussed below may be used depending on whether there are obstacles in the line ahead or not, and whether a small or a large obstacle is encountered.

Double Centering

The method used to accomplish the extension is known as DOUBLE CENTERING or DOUBLE REVERSING. Suppose you are prolonging the line AB shown in figure 13-16. You set up the transit at B, backsight on A, plunge the telescope so as to sight ahead, and set the marker



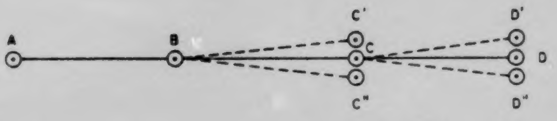
ANGLES AROUND STA A BY REPETITION

Point Sighted	No Rep	Plate Reading	Mean Angle
B		00°00'	
C	1	82°45'	
	6	136°28'	82°44'40"
C		00°00'	
B	1	277°15'	
	6	223°32'	277°15'20"
		Sum	360°00'00"
		Closure	00°00'00"

transit #2

Smith, J., EA 2, ⊼
Jones, R., EA CN, notes
6 June 19 -clear-warm

45.7.9
Figure 13-15.—Notes for 6-times angles around a point.



45.102
Figure 13-16.—Double centering.

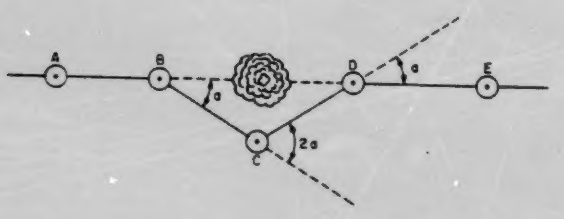
at C'. Now with the telescope still inverted, you again sight back on A, but this time by rotating the telescope through 180°. You then replunge the telescope and mark the point C". Mark the point C halfway between C' and C". This is the required point on the line AB extended. If the instrument is in perfect adjustment (which is seldom, if ever, the case) points C' and C" will coincide with point C. For further extension the instrument is moved to C and the procedure repeated to obtain D.

Bypassing Small Obstacles

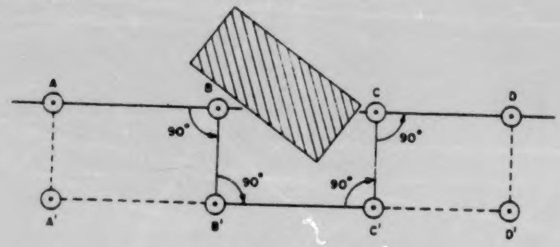
When a tree or other small obstacle is encountered in the line of sight between two points, the solution below should be used. This procedure is also known as ANGLE OFFSET method. The transit or theodolite is set up at point B (fig. 13-17), as far from the obstacle as practical. Point C is set off the line, near the obstacle, and where the line BC will clear the obstacle. At B, measure the deflection angle a. Move the instrument to C, and lay off the deflection angle 2a. Measure the distance BC, and lay off the distance CD equal BC. Move the instrument to D, and lay off the deflection angle a. Mark the point E. Then line DE is the prolongation of the line AB.

Bypassing Large Obstacles

When a line is being run between two fixed points and a large obstacle, such as a building is encountered, there are numerous methods that can be used to extend the line beyond the obstacle. The commonly used method to solve this problem is shown in figure 13-18 which is also known as the PERPENDICULAR OFFSET method. The solution establishes a line parallel to the original line and at a distance to clear the obstruction. Once this parallel line passes the obstruction, another parallel line is established using the same distance value in the other direction. This second parallel line is the extension of the original line.



45.103(45B)
Figure 13-17.—Bypassing obstacles.



45.104(45B)
Figure 13-18.—Bypassing a large obstacle.

The instrument is set up at B, figure 13-18, and 90° angle turned from line AB. The distance BB' is carefully measured and recorded. The instrument is moved to B' and another 90° angle turned. B'C' is laid off to clear the obstacle. The instrument is moved to C' and a third 90° angle turned. Distance CC' equal to BB' is measured and marked. This establishes a point C, on the original line. The instrument is moved to C and a fourth 90° angle is turned to establish the alignment CD which is the extension of AB beyond the obstacle.

When the obstacle-clearing distance, BB' or CC', is less than a tape length, the turning of four 90° angles can be avoided. Perpendicular offsets may be erected from points A and B (fig. 13-18); AA' equals BB'. Set up the instrument at B', perhaps measuring angle A'B'B to check if it's 90°. Extend line A'B' to C' then to D'—making sure that point C' clears the obstacle. Then layoff perpendicular offset C'C equal to AA' or BB' and perpendicular offset D'D equal to C'C. Then line CD is the extension of line AB. The total distance of the line AD is the sum of the distances AB, B'C' and CD.

You may also compute the diagonals formed by the end rectangles and compare the result to actual measurement if feasible as a further check.

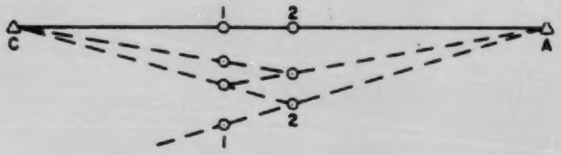
Line Between Non-Intervisible Points

It is sometimes necessary to run a straight line between non-intervisible points under circumstances which make the previously described methods of bypassing an obstacle unfeasible. If there is an intermediate point on the straight line from which both of the end points can be observed, the method called "balancing in" (also called "bucking" in, "jiggling" in, or "wiggling" in) may be used.

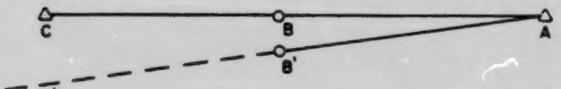
A problem often encountered in surveying is to find a point exactly on the line between two other points when neither can be occupied, or when an obstruction such as a hill lies between the two points. The point to be occupied must be located so that both of the other points are visible from it. The process of establishing the intermediate point is known as WIGGLING IN or RANGING IN.

The approximate position of the line between the two points, at the instrument station is first estimated by using two range poles. The range poles are lined in alternately in the following manner. In figure 13-19A, range pole 1 is set and range pole 2 is moved until it is exactly on line between pole 1 and point A. This is done by sighting along the edge of pole 1 at the station A until pole 2 seems to be on line. Range pole 2 is set and pole 1 is moved until it is on line between pole 2 and point C. Now pole 2 is moved into line again and then pole 1 alternately until both are on line AC. The line will appear to pass through both poles and both stations from either viewing position.

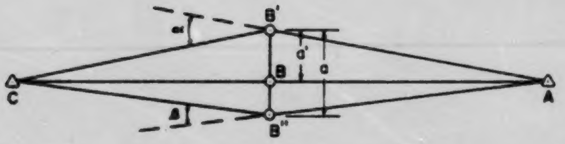
After finding the approximate position of the line between the two points, set up the instrument on this line. The instrument probably will not be exactly on line, but will be over a point such as B', in view B, fig. 13-19. With the instrument at B', backsight on A, and plunge the telescope and notice where the line of sight (C') passes the point C. Estimate this distance (CC'), and also the distance that B' would be away from C and A. Estimate the amount to move the instrument to place it on the required line. Thus if B' is midway between A and C, and C' misses C by 3 ft. to the left, B' must be moved about 1.5 ft. to the right to reach B. Continue the sequence of backsighting, plunging the telescope, and moving the instrument until the line of sight passes through both A and C.



(A) Ranging-in.



(B) Wiggling-in on line.



(C) Measuring a point on line.

45.769

Figure 13-19.—Setting up on a line between two points.

During this procedure, the telescope is reversed but the instrument is not rotated, that is, if the telescope is reversed for backsighting on A, all sightings on A are made with telescope reversed. Mark a point on the ground directly under the instrument. Then, repeat the procedure with the telescope direct for each backsight on A. Mark a second point on the ground. The required point on the line AC is then the midpoint between the two marked points.

The procedure outlined above is usually time consuming. Even though the shifting head of the instrument is used in the final instrument movements, the instrument may have to be picked up and moved several times. The following procedure often saves time. After finding the approximate position of the line between the two points, two points (B' and B'', view C, fig. 13-19) are marked 1 or 2 feet apart where they are known to straddle the line AC. Set up over each of these two points in turn, and measure the deflection angles α and β. Also measure the horizontal distance a, between points B' and B''. Then the position (B) on the line AC can be found by using the following equation:

$$a' = a \frac{\alpha}{\alpha + \beta}$$

in which a' is the proportionate offset distance from B' toward B'' for the required point B, and α and β are both expressed in minutes, or in seconds.

Random Line

When it is necessary to locate intermediate stations on a survey line, the random line method may be used. In figure 13-20, stations 0 + 00 and 2 + 50, now separated by a grove of trees, were placed at some time in the past. You are required to locate stations 1 + 00 and 2 + 00 which lie among the trees.

Run a line at random from station 0 + 00 until you can see station 2 + 50 from some point A, on the line. The transitman measures the angle at A and finds it to be 108° 00'. The distances from A to stations 0 + 00 and 2 + 50 are chained and found to be 201.00' and 98.30' respectively. With this information it is now possible to locate the intermediate stations between stations 0 + 00 and 2 + 50. The distances AB and AD can be computed by similar triangles method as follows:

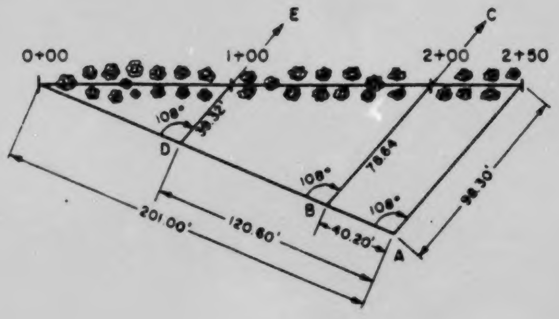
$$AB = \frac{50}{250} \times 201.0 = 40.20'$$

$$\text{and } AD = \frac{150}{250} \times 201.0 = 120.60'$$

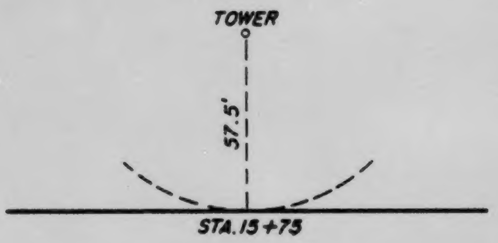
These distances are laid off on the random line from point A toward station 0 + 00. The instrumentman then occupies points B and D and turns the same angle (108° 00') that he measured at point A and establishes points C and E on lines from points B and D through the sought stations. The distances to those stations are then chained from the random line. These distances are computed by similar triangles method as follows:

$$\text{B to station 2 + 00 (BC)} = \frac{200}{250} \times 98.3' = 78.64'$$

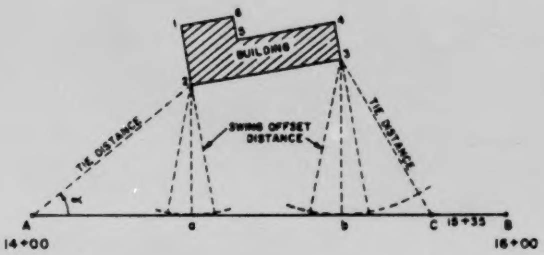
$$\text{D to station 1 + 00 (DE)} = \frac{100}{250} \times 98.3' = 39.32'$$



45.770
Figure 13-20.—Random line.



45.107
Figure 13-21.—Tying in a point by perpendicular offset.



45.771
Figure 13-22.—Swing offset.

TIE IN

Fundamentally, the purpose of traversing is to determine the locations of a framework of primary horizontal-control points of known horizontal location, from which the horizontal locations of other points not on the traverse can be determined, or from which other points not on the traverse can be "set" or "established" in given horizontal locations.

Determining the location of a point with reference to a station, or to two stations, on a traverse is commonly called "tying in" the point. The simplest way to tie in a point is by "perpendicular offset" as shown in figure 13-21. The various procedures for tying in are described below.

Swing Offsets

The swing-offset method is used for locating points close to the control lines (see fig. 13-22). Measurement of a swing-offset distance provides an accurate determination of the perpendicular distance from the control line to the point being located. It is somewhat similar to the range tie (explained below) but as a rule requires no angle measurement. In determining the swing-offset distance, one tapeman holds the zero mark of the tape at a corner of the structure while the other tapeman swings an arc with the tape to the transit line (AB). When the shortest reading on the graduated end of the tape is observed, the swing-offset or perpendicular distance to the control line is obtained (at points a or b). The alinement of AB can be first established at the intersection of the offset points and the tapeman can read the value. The more common practice is to have the instrumentman read the shortest distance through the telescope. The distance from the instrument station to the swing-offset points (a or b) is measured. A tie or check distance can be measured from some known point (c) along the line or an angle α can be read for a tie from either instrument station.

Perpendicular Offsets

The method of perpendicular offsets from a control line (fig. 13-23) is similar to swing offsets, but is more suitable for locating detail of irregular objects, such as stream banks and winding roads. The control line is established close to the irregular line to be located, and perpendicular offsets (aa', bb', cc' and so on) are measured to define the irregular shape. When the offset distances are short, the 90° angles are usually estimated, but when the distances are several hundred feet long, the angles should be laid off with an instrument.

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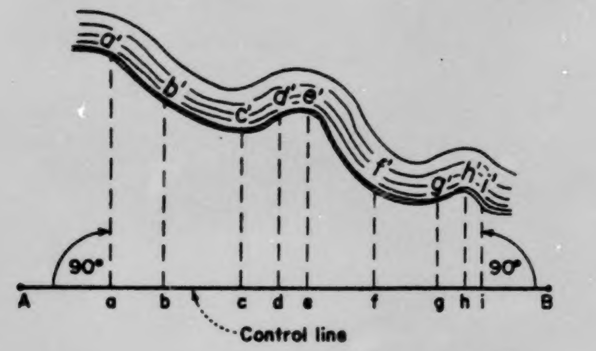

45.772
Figure 13-23.—Perpendicular offsets.

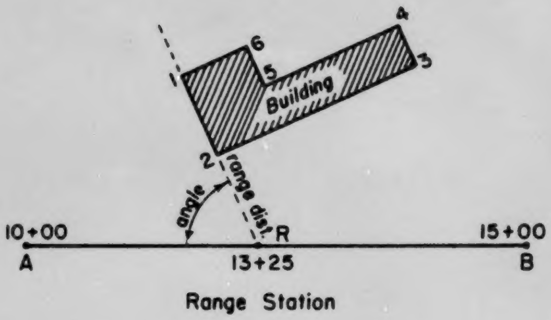

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Figure 13-24.—Ranging ties.

The distances to the offset points (from a to i) are measured along the control line.

Range Ties

A range tie is another method of determining a point's location using an angle and a distance. The method requires extra instrument manipulation and should be used only when none of the previous methods can be used satisfactorily. Actually, the method will establish not only the corner of a structure, but also the alinement of one of the sides. In figure 13-24 assume that the building is not visible from either A or B, or that either or both of the distances from A or B to a corner of the building cannot be measured easily. With the instrument set up at either A or B and the line AB established, one member of the party moves along AB until he reaches point R which is the intersection of line 1-2 extended. The instrument is moved and set up on R and the distance along the line AB to R is measured. An angle measurement to the building is made using either A or B as the backsight. The range distance (R-2) is measured as well as the building dimensions.

SETTING POINTS

When you "tie in" a point adjacent to a traverse, you measure distances and turn angles to determine what might be called the "tie data" of the point. To "set" a point adjacent to a traverse means to establish the location of a point in accordance with given "tie data". Again the "tie data" may be a perpendicular offset distance from a specified traverse station, angle and distance from a specified station, distance from each of two stations, angles from two stations, or angle from one station and distance from the other.

To set a point in accordance with its angle and distance from a single station you simply set up the transit at the station, turn the designated angle, and chain the designated distance along the line of site. For "perpendicular offset", of course, the angle is 90°.

To set a point in accordance with distance from each of two stations you can manage by using two tapes, provided each of the distances is less than a full tape length. Suppose, for example, you want to set a point which is to be 75 ft from station 0 + 75 and 50 ft from station 1 + 75. Have the zero end of one 100-ft tape held on station 0 + 75 and the zero end of another on station 1 + 75. Run out the tapes and place the 75-ft mark on the first in contact with the 50-ft mark on the second. When both tapes are drawn taut, the place of contact between them will be over the designated location of the point.

If one or both of the distances is greater than the length of a tape, you can determine the direction of one of the tie lines by appropriate triangle solution. In figure 13-25 you want to set point B 120.0 ft from station A and 83.5 ft from station C. A and C are 117.0 ft apart. You can determine the size of the angle at A by triangle solution as follows:

$$1 - \cos A = \frac{2(s - b)(s - c)}{bc}$$

$$s = 1/2(120.0 + 117.0 + 83.5) = 160.25$$

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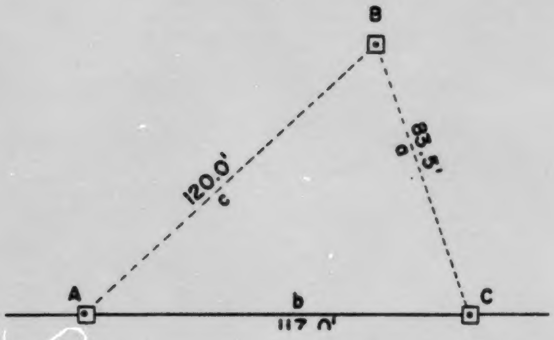


45.670
Figure 13-25.—Locating a point by distances from two stations.

$$1 - \cos A = \frac{2(43.25)\,(40.25)}{(117.0)\,(120.0)} = 0.24797$$

$$\cos A = 1.00000 - 0.24797 = 0.75203$$

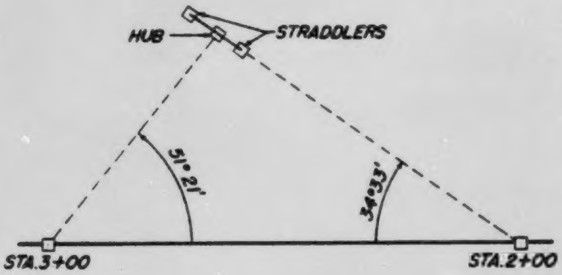
$$A = 41^\circ\ 14'$$

To set point B, then, you can set up a transit at A, sight on C, turn 41° 14' to the left, and measure off 120.0 ft on that line of sight. To check, then, you can measure BC to ensure that it measures 83.5 ft.

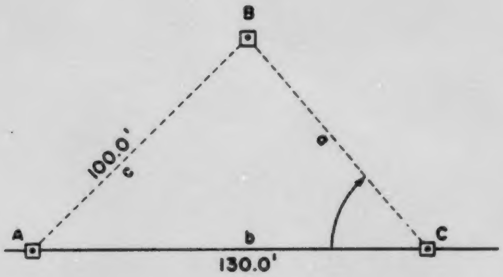
To set a point in accordance with its angle from each of two traverse stations it is customary to use a pair of "straddle hubs" (familiarly called "straddlers") as shown in figure 13-26. Here the point was to be located at an angle of 34° 33' from station 2 + 00 and at an angle of 51° 21' from station 3 + 00. The transit was set up at station 2 + 00, sighted on station 3 + 00, and an angle of 34° 33' was turned to the right. On this line of sight a pair of straddle hubs was driven, one on either side of the estimated point of intersection of the tie lines. A cord was stretched between the straddlers.

The transit was then shifted to station 3 + 00, sighted on station 2 + 00, and an angle of 51° 21' was turned to the left. A hub was driven at the point where this line of sight intercepted the cord between the straddlers.

To set a point in accordance with its angle from one station and distance from the other it is best to determine the direction of the distance line by triangle solution. In figure 13-27 point B is to be located 100.0 ft from station A and at an angle of 50° 00' from station C. You can determine the size of the angle at A by first determining the size of angle B, then subtracting the sum of angles B and C from 180°. The solution for angle B is as follows:



45.113
Figure 13-26.—Setting a point by the use of straddlers.



45.671
Figure 13-27.—Locating a point by angle and distance from two stations.

$$\sin B = \frac{130.0\ \sin\ 50^\circ\ 00'}{100.0}$$

$$\sin B = \frac{130.0\ (0.76604)}{100.0} = 0.99585$$

Angle B, then, measures, to the nearest minute, 84° 47'. It follows that angle A measures 180° 00' - (84° 47' + 50° 00'), or 45° 13'. Set up a transit at A, sight on C, and turn 45° 13' to the right. Then set B by measuring off 100.0 ft on this line of sight. As a check, set up the transit at C, sight on A, turn 50° 00' to the right, and ensure that this line of sight intercepts the marker at B.

TRANSIT-TAPE SURVEY PROCEDURE

The procedure followed in a particular transit-tape survey will vary in accordance with the nature of the survey, the policy followed in the battalion, and the preferences of the party chief. This course can only describe customary procedure in very general terms.

SELECTING POINTS FOR MARKING

All points where a traverse changes direction are marked, usually with a hub which locates the station exactly, plus a "guard stake" on which the station of the change-of-direction point is inscribed (such as 12 + 35). In the expression "station 12 + 35" the 12 is called the "full station" and the 35 is called the "plus."

The points which are to be tied to the traverse, or set in the vicinity of the traverse, are usually selected and marked or set as the traverse is run. The corresponding tie stations on the traverse are, of course, selected and marked at the same time. The first consideration in selecting tie stations is VISIBILITY—meaning that the tie stations and the point to be tied or set must be intervisible. The next is PERMANENCY (not easily disturbed). Then lastly, the strength of intersection, which means, in general, that the angle between two tie lines should be as close to 90° as possible. The more acute or obtuse the angle between tie lines is, the less accurate will be the location of the point defined by their intersection.

PARTY PERSONNEL

A typical transit-tape survey party contains two chainmen, a transitman, a recorder (sometimes the transitman or party chief doubles as recorder), a party chief (who may serve as instrumentman and/or recorder), and axmen as needed. The transitman carries, sets up, and operates the transit; the chainmen do the same with the tapes and the marking equipment.

When the transitman turns an angle, he calls out the identity and size of the angle to the recorder, as: "Deflection angle AB to BC, 75° 16', right." The recorder repeats this back, then makes the entry. Similarly, the head chainman calls out the identity and size of a linear distance, as: "B to C, 265.72 ft;" whereupon the recorder repeats this back and then makes the entry. If the transitman closes the horizon around a point he calls out: "Closing angle, so-and-so." The recorder repeats this back, and then adds the closing angle to the original angle. If the sum doesn't come close to 360°, the recorder notifies the party chief.

The party chief is in complete charge of the party, and makes all the significant decisions—such as, for example, which stations are to be marked on the traverse.

PRESCRIBED ORDER OF PRECISION

The important distinction between "accuracy" and "precision" in surveying was explained in chapter 11. In general, any survey must be carried out accurately—meaning that in any survey mistakes must be avoided. The precision of a survey, however, depends upon the "order of precision" which is either specified or to be inferred from the nature of the survey.

The various orders of precision are absolute, not relative in implication. Federal agencies control surveys, such as those made by USC&GS are generally classified into four orders of precision, namely: FIRST ORDER, SECOND ORDER, THIRD ORDER, and FOURTH ORDER control surveys. The FIRST ORDER being the highest and the FOURTH ORDER the lowest standard of accuracy. Some authorities insist on using six orders of precision, but at present, even the fourth order is seldom used because of the increase in value of real estate and the availability of precise ordinary surveying instruments.

Because of the type of instruments available to you in the SEABEES, most of your surveys may not require a precision higher than third order survey. When the order of precision is not specified, you may use table 13-1 as a standard for a horizontal control survey when using the traverse control method. For surveys that call for higher order of precision, you will have to use theodolites to obtain the required precision. (See chapter 9 of this training manual).

Triangulation control procedures is discussed fully in Engineering Aid 1 & C. At present however, you may encounter survey problems that may require the use of the triangulation method. In such a case, you may use table 13-2 as a guide for order of precision, if not otherwise specified in the survey.

For other survey precisions, you may refer to Federal Board of Surveys and Mapping publications, Surveying and Mapping, "Classification and Standards of Accuracy of Geodetic Surveys," Volume 19, No. 2, pp. 219-224 (when necessary).

45.774

Table 13-1.–Control traverse order of precision

ORDER OF PRECISION	MAXIMUM NUMBER OF AZIMUTH COURSES BETWEEN AZIMUTH CHECKS	DISTANCE MEASUREMENT ACCURATE WITHIN	MAXIMUM LINEAR ERROR OF CLOSURE	MAXIMUM ERRORS OF ANGLES
FIRST ORDER	15	$\frac{1}{35,000}$	$\frac{1}{25,000}$	* 2 sec $\sqrt{N}$ or 1.0 sec per station
SECOND ORDER	25	$\frac{1}{15,000}$	$\frac{1}{10,000}$	* 10 sec $\sqrt{N}$ or 3.0 sec per station
THIRD ORDER	50	$\frac{1}{7,500}$	$\frac{1}{5,000}$	* 30 sec $\sqrt{N}$ or 8.0 sec per station
FOURTH ORDER	--	$\frac{1}{3,000}$	$\frac{1}{1,000}$	2 min or compass

N = the number of stations carrying azimuth.
* Use whichever is smaller in value.

45.775

Table 13-2.—Triangulation order of precision

PRECISION	APPLICATION	BASE LINE MEASUREMENT MAX. PROBABLE ERROR	TRIANGLE CLOSURE: MAX. AVERAGE ERROR	LENGTH CLOSURE: MAX. DISCREPANCY BET. MEASURED AND COMPUTED LENGTH BASE LINE
FIRST ORDER	CASE I For city and scientific study survey.	$\frac{1}{1,000,000}$	1.0 sec	$\frac{1}{100,000}$
	CASE II Basic netword of U.S.	$\frac{1}{1,000,000}$	1.0 sec	$\frac{1}{50,000}$
	CASE III All other purposes.	$\frac{1}{1,000,000}$	1.0 sec	$\frac{1}{25,000}$
SECOND ORDER	CASE I Area networks and supplemental cross arcs in national net.	$\frac{1}{1,000,000}$	1.5 sec	$\frac{1}{20,000}$
	CASE II Coastal areas, inland waterways, and engineering surveys.	$\frac{1}{500,000}$	3.0 sec	$\frac{1}{10,000}$
THIRD ORDER	Topographic Mapping	$\frac{1}{250,000}$	5.0 sec	$\frac{1}{5,000}$

The practical significance of a prescribed or implied order of precision lies in the fact that the instruments and methods used must be those which are capable of attaining the required precision. The precision of an instrument is indicated by a fraction in which the numerator is the inherent error (in a 1-minute transit the inherent error is 1 minute) and the denominator the total number of units in which the error occurs. For a transit this last is 90°, or 5400'. The precision of a 1-minute transit, then, is 1/5400, or about adequate for a third-order survey.

Precision of a tape is given in terms of the inherent error per 100 ft. A tape which can be read to the nearest 0.01 ft has a precision of 0.01/100, or 1/10,000—adequate for second-order work.

PRECISION FROM LINEAR ERROR OF CLOSURE

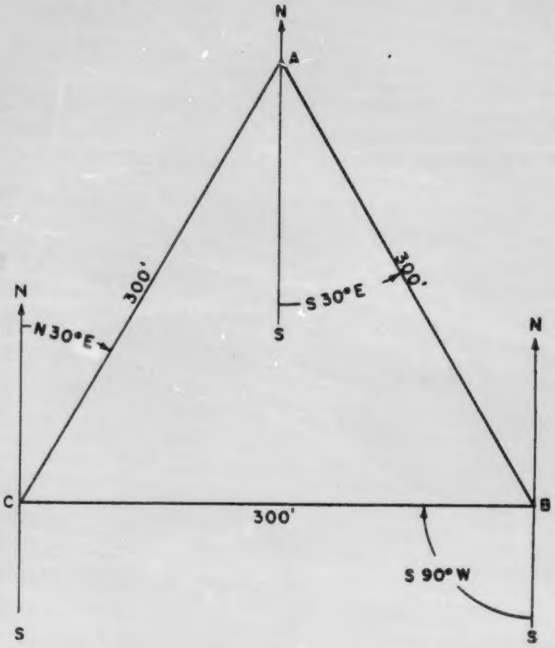
For a closed traverse you must attain a "ratio of linear error of closure" which corresponds to the order of precision prescribed or implied for the traverse. The ratio of linear error of closure is a fraction in which the numerator is the linear error of closure and the denominator the total length of the traverse.

To understand the concept of linear error of closure, consider the closed traverse shown in figure 13-28. Beginning at station C, this traverse runs N 30° E 300 ft, thence S 30° E 300 ft, thence S 90° W 300 ft. The end of the closing traverse (BC) lies exactly on the point of beginning (C), indicating that all angles were turned and all distances chained with perfect accuracy, resulting in "perfect closure", or an error of closure of 0 ft.

Now, perfect accuracy in measurement is in the nature of things impossible. Therefore, in a real-life survey the end of the closing traverse BC, would be likely to lie some distance away from the point of beginning (C). Suppose that this was a horizontal linear distance of 0.09 ft. The total length of the traverse is 900.00 ft. The ratio of error of closure, then, is 0.09/900.00, or 1/10,000. This is equivalent to the precision prescribed for second-order work.

PRECISION FROM MAXIMUM ANGULAR ERROR OF CLOSURE

You know that the sum of the interior angles of a closed traverse should theoretically equal the product of 180°(n - 2), n being the number of sides in the polygon described by the traverse. A prescribed "maximum angular error of closure" is stated in terms of the product of a given angular value times the square root of the number of interior angles in the traverse. Suppose, for example, that for the traverse shown in figure 13-28 the prescribed maximum angular error of closure is 01'$\sqrt{3}$, there being 3 interior angles in the figure. The sum of the interior angles should be 180°. Suppose that the sum of the angles, as actually measured and recorded, was 179° 57'. The angular error of closure is 03'. The maximum permissible is the product of 01' times the square root of 3, or about 1.73'. You have therefore exceeded the prescribed maximum angular error of closure.



23.240.1A

Figure 13-28.—Closed traverse with perfect closure.

PRECISION SPECIFICATIONS

The following specifications are intended to give you only a general idea of the typical precision requirements for various types of transit-tape surveys. Where linear and angular errors of closure are specified, it is understood that a closed traverse is involved.

For many types of preliminary surveys, and for land surveys in which the land is relatively inexpensive, typical precision specs might read as follows:

> Transit angles to nearest minute, measured once. Sights on range poles plumbed by eye. Tape leveled by eye, with standard tension estimated. No temperature or sag corrections. Slopes under 3 percent disregarded. Slopes over 3 percent measured by breaking chain, or by chaining slope distance and applying calculated correction. Maximum angular error of closure, 1.5' $\sqrt{n}$. Maximum ratio linear error of closure, 1/1000. Pins or stakes set to nearest 0.1 ft.

For most land surveys and highway location surveys, typical precision specifications might read as follows:

> Transit angles to nearest minute, measured once. Sights on range poles, plumbed carefully. Tape leveled by hand level, with standard tension by tensionometer or sag correction applied. Temperature correction applied if air temperature more than 15° different from standard (68° F). Slopes under 2 percent disregarded. Slopes over 2 percent measured by breaking chain or by applying approximate slope correction to slope distance. Pins or stakes set to nearest 0.05 ft. Maximum angular error of closure 1' $\sqrt{n}$. Maximum ratio linear error of closure, 1/3000.

For important boundary surveys and extensive topographical surveys, typical precision specifications might read as follows:

> Transit angles by 1-minute transit, repeated 4 times. Sights taken on plumb lines or on range poles carefully plumbed. Temperature and slope corrections applied; tape leveled by level. Pins set to nearest 0.05 ft. Maximum angular error of closure, 0.5' $\sqrt{n}$. Maximum ratio linear error of closure, 1/5000.

Note that in the first two specifications one-time angular measurement is considered sufficiently precise. Many surveyors, however, use two-time angular measurement as a customary procedure, to maintain a constant check on possible mistakes.

ANGLES VS. DISTANCES

It is usually the case on a transit-tape survey that the equipment for measuring angles is considerably more precise than the equipment for measuring linear distances. This fact leads many surveyors into a tendency to measure angles with great precision while overlooking important errors in the measurement of linear distances.

Now, it is useless to make the precision of angular measurement greater than that of linear measurement, because your angles are, you might say, only as good as your linear distances. Suppose, for example, that you are running traverse line BC at a right deflection angle of 63° 45' from AB, 180.00 ft to station C. You set up at B, orient the telescope to AB extended, and turn exactly 63° 45' 00" to the right. But then, instead of measuring off 180.00 ft, you measure off 179.96 ft. Regardless of the precision with which you turn all of the subsequent angles in the traverse, every station will be mislocated because of the error in the linear measurement of BC.

You can see, then, that angles and linear distances must be measured with the same precision, and that it is useless to measure angles more precisely than you measure distances.

ERRORS AND MISTAKES IN TRANSIT WORK

In chapter 11 of this training manual, we have discussed the difference between errors and mistakes. It was stated that ERRORS are caused by the effects of nature, by the condition of the instrument, and the physical condition of the personnel performing the survey, whereas MISTAKES are plain human "blunders."

In transit work, errors are grouped into three general categories, namely: INSTRUMENTAL, NATURAL and PERSONAL errors. We will discuss first these errors and the common MISTAKES in transit work will be explained later.

TRANSIT INSTRUMENTAL ERRORS

A transit will not measure angles accurately unless the following conditions exist in the instrument.

1. The vertical crosshair must be perpendicular to the horizontal axis. If the vertical crosshair is not thus perpendicular, measurement of horizontal angles will be inaccurate.
2. The axis of each of the plate levels must be perpendicular to the vertical axis. If this is not the case, the instrument cannot be accurately leveled. If the instrument is not level, measurement of both horizontal and vertical angles will be inaccurate.
3. The line of sight through the telescope must be perpendicular to the horizontal axis. If it is not, the line of sight through the telescope inverted will not be 180° opposite the line of sight through the telescope erect.
4. The horizontal axis of the telescope must be perpendicular to the vertical axis. If it is not, the measurement of both horizontal and vertical angles will be inaccurate.
5. The axis of the telescope level must be parallel to the line of sight through the telescope. If it is not, the telescope cannot be accurately leveled. If the telescope cannot be accurately leveled, vertical angles cannot be accurately measured.
6. The point of intersection of the vertical and horizontal crosshairs must coincide with the true optical axis of the telescope. If it doesn't, measurement of both horizontal and vertical angles will be inaccurate.

Any or all of the above conditions may be absent in an instrument, because of maladjustment, damage, or defect. Procedures for correcting maladjustments are described in chapter 10 of this training manual.

NATURAL ERRORS

Common causes of natural errors in transit work are as follows.

1. Settlement of the tripod in yielding soil. If the tripod settled evenly—that is, if the tip of each leg settled precisely the same amount—there would be little or no resulting error in the measurement of horizontal angles. Settlement is usually uneven, however, which results in dis-levelment of the instrument.
2. Refraction—but the effect of this is usually negligible in ordinary-precision surveying.
3. Unequal expansion or contraction of instrument parts, caused by excessively high or excessively low temperature. For ordinary-precision surveying the effect of this is also usually negligible.
4. High wind, which may cause plumbing errors when plumbing with plumb bob and cord, and reading errors because of vibration of the instrument.

PERSONAL ERRORS

Personal errors are the combined results of carelessness and of the limitations of the human eye in setting up and leveling the instrument and in making observations.

Common causes of personal errors in transit work are as follows:

1. FAILURE TO PLUMB THE VERTICAL AXIS EXACTLY OVER THE STATION. Figure 13-29 shows how the resulting inaccuracy increases drastically as the sight distance decreases. In that figure, an instrument supposed to be set up at A was actually set up at A', 40 ft away from A. (For demonstration purposes the figure was exaggerated to magnify the error; in actual occurrence the eccentricity amounts only to a fraction of an inch. Remember that mathematically, 1 inch is the arc of 1 minute when the radius is 300 ft.)

In the upper view you can see that with B located 300 ft from A, the angular error caused by the displacement is about 8°. In the lower

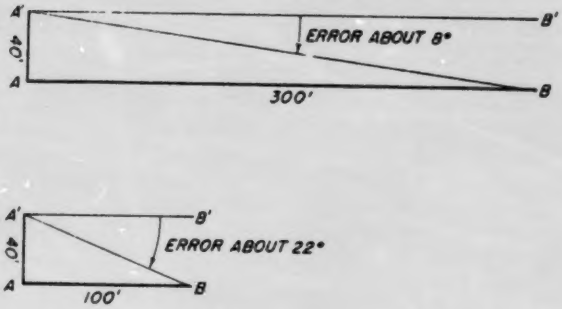


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Figure 13-29.—Exaggerated illustration of error caused when the transit is not centered exactly over the station.

view, however, with B located only 100 ft from A, the angular error caused by the displacement is about 22°.

The practical lesson to be learned from this is that you must plumb the instrument much more carefully for a short sight than for a long one.

2. Failure to center plate level bubbles exactly. The result of this is that the instrument is not leveled exactly. The consequent error is at a minimum for a horizontal sight, and increases as a sight becomes inclined.

The practical lesson is that you should level the instrument much more carefully for an inclined sight than for a horizontal one.

3. Inexact setting or reading of a vernier. The use of a small, powerful pocket magnifying glass is very helpful here. Also, when you have determined which vernier graduation most nearly coincides with a limb graduation, it is a good idea to check your selection by examining the graduations on either side of the one selected. These should fail of coincidence with their limb counterparts by about the same amount.

4. Failure to line up the vertical crosshair with the true vertical axis of the object sighted. The effect is similar to that of not plumbing exactly over the station—which means that the error increases drastically as the length of the sight decreases.

5. Failure to bring the image of the crosshair and/or that of the object sighted into clear focus (PARALLAX). A fuzzy outline makes exact alignment difficult.

Common mistakes in transit work are as follows:

1. TURNING THE WRONG TANGENT SCREW; for example, turning the lower tangent screw AFTER taking a backsight will introduce an error in the backsight reading.

2. FORGETTING TO TIGHTEN THE CLAMPS, OR THE SLIPPING OF A CLAMP WHEN IT IS SUPPOSED TO BE TIGHT.

3. READING IN THE WRONG DIRECTION FROM THE INDEX (zero mark) ON A DOUBLE VERNIER.

4. READING THE WRONG VERNIER, that is, reading the vernier opposite the one which was set.

5. READING ANGLES IN THE WRONG DIRECTION, that is, reading from the outer row rather than the inner row or vice versa, on the horizontal scale.

6. FAILURE TO TAKE A FULL SCALE READING BEFORE READING THE VERNIER. Dropping the 20 or 30 minutes from the reading, erroneously recording only the number of minutes indicated on the vernier, such as 15° 18' instead of 15° 48'. (Do not be so intent on reading the vernier that you lose track of the full scale reading of the circle.)

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CHAPTER 15

TOPOGRAPHIC SURVEYING AND MAPPING

Topography refers to the characteristics of the land surface. These characteristics include RELIEF, NATURAL FEATURES, and MAN-MADE features. Relief is the configuration of the earth's surface and includes such features as hills, valleys, plains, summits and depressions and other natural features such as trees, streams, lakes, and the like. Man-made features are such as highways, bridges, dams, wharfs, or buildings.

A graphic representation of the topography of an area is called a "topographic map." A topographic map is simply a drawing which shows the natural and artificial features of an area. A "topographic survey" is a survey conducted for the purpose of obtaining the data required for the preparation of a topographic map. Generally speaking, this data consists essentially of the horizontal and vertical locations of the features which are to be shown on the map. The horizontal location of a feature is called its "location in plan;" the vertical location is called the "location in elevation."

The field work in a topographic survey consists principally of (1) the establishment of a basic framework of horizontally and vertically located control points (called "instrument points" or "stations"), and (2) the determination of the horizontal and vertical locations of "details" in the vicinity of each instrument point.

ESTABLISHING TOPOGRAPHIC CONTROL

Topographic control consists of two parts (1) horizontal control and (2) vertical control. The horizontal control means the locations of the stations and monuments from which topographic details are located in the plans. The vertical control consists of the bench marks from which the elevations are obtained. Actually vertical control points are established by running a line of levels. Control points of the lowest order of precision may be established by some other means.

When the area to be mapped is very large, you must make every effort to eliminate the cumulative effects of errors. The first step is to establish the most important or primary control stations. This is usually accomplished fairly accurately by triangulation. Then starting at any primary control station, run traverses to establish secondary control stations. Close each traverse on one of the primary stations. Still other stations may be run by the plane table method or by the compass method. Thus you can see that the primary control stations form a framelike network upon which the rest of the survey is built. Errors, therefore, cannot accumulate any further than the distance between the primary stations.

Aerial photography sometimes is used for surveying large tracts of land. With the help of sensitive instruments, practical topographic maps may be made from photographs taken from airplanes. This method has been used for such jobs as traffic studies, city planning, locating pipelines, railroads, dam sites, and airfields.

HORIZONTAL CONTROL

The selection and horizontal location of a number of conveniently located instrument points or stations, from which details may be located, may be accomplished by traversing, by triangulation, or by the combined use of both methods. On an important, large-area survey there may be both "primary" control, in which a number of widely separated primary control points are located with a high degree of precision; and "secondary" control, in which stations are located with less precision within the framework of the primary control points.

If horizontal control is established by traversing, there may be both primary and secondary traverses run. A primary traverse, which locates the primary control points, follows routes which will produce conveniently located stations. Such routes might run along roads, ridges, valleys, edges of wooded areas, public land lines, or (particularly for small areas) near the perimeter of the tract.

If all the details in the area can be conveniently located from stations on the primary traverse, no secondary traverses are required. Generally, however, the size and/or character of the terrain make it necessary that control points in addition to those on the primary traverse be established. These points are located on secondary traverses called "cross-ties," each of which is run across the area from one primary traverse station to another.

Consider, for example, the situation shown in figure 15-1. This figure shows a tract, bounded on three sides by highways and on the fourth side by a fence. For simplification, the figure shows only the items to be discussed in the following explanation; an actual complete plan would include a title, date, scale, north arrow, and so on.

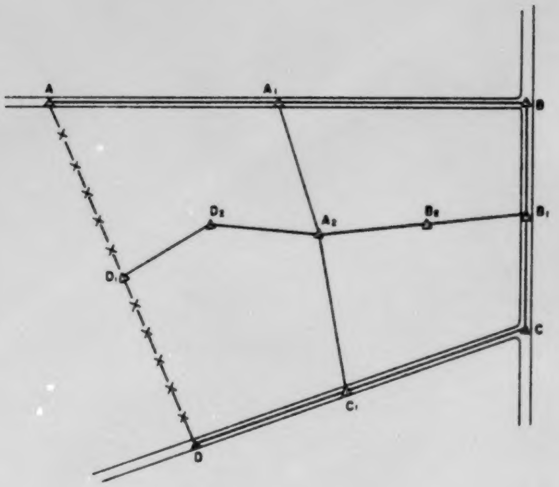
The primary traverse ABCD runs around the perimeter of the tract. It might be the case that details on the whole tract could be satisfactorily located from control points on the primary traverse — that is, from stations on the highways, such as A, A_1, B, B_1, C, C_1, and D; and from one or more stations along the fence, such as D_1. However, the size and character of the terrain might make it necessary to establish instrument points within the perimeter of the tract, such as D_2, A_2, and B_2. To establish these stations, secondary cross-tie traverses would be run as shown.

Important details are located both horizontally and vertically from each station. Minor details may be located horizontally only for plotting purposes.

Field notes for the survey sketched in figure 15-1 would contain (1) notes showing the horizontal locations of the stations, and (2) level notes for determining the elevations of the stations.

VERTICAL CONTROL

As stated in chapter 14 of this training manual, the network of benchmarks is established for vertical control. In topographic surveying, these benchmarks will serve as (1) starting points and closure for the leveling operations when locating details, and (2) as a ready reference elevation for subsequent construction work. Also, the established horizontal control marks are often used as the benchmarks, because the level routes generally follow the traverse lines. Vertical control is accomplished by direct leveling; however, it may be done by trigonometric leveling for a limited area or in rough terrain. The datum for some surveys may be assumed, but it is seldom the case that a precisely established benchmark is not available in the area, specially within the United States. The surveyor must make every effort to tie-in his surveys to these marks, if feasible, for proper location and identification.



45.404
Figure 15-1. — Primary traverse and secondary traverse.

Four degrees of precision are commonly used in establishing vertical control in a topographic survey for an intermediate-scale map.

1. 0.05 ft $\sqrt{\text{distance in miles}}$. You can see that this is equivalent to third order precision given in table 14-2. This is used as the standard for surveys in very flat region and when the contour interval is 1 ft or less, or on surveys which require the determination of gradient of streams, or in surveys to establish the grades for proposed drainage and irrigation systems.

2. 0.1 ft $\sqrt{\text{distance in miles}}$. This is equivalent to fourth order precision. It is used in a survey where the contour interval of the map is 2 feet.

3. 0.3 ft $\sqrt{\text{distance in miles}}$. This is used for a contour interval of 5 feet.

4. 0.5 ft $\sqrt{\text{distance in miles}}$. This is used for a contour interval of 10 ft and may be accomplished by stadia leveling, a method which is very advantageous in hilly terrain.

Precisions 1 and 2 above are used for a large-scale map which generally has a contour interval of 1 or 2 ft. For an extensive survey of a large area, No. 1 is used; No. 2 is used for surveys of smaller areas.

Vertical controls are also necessary for hydrographic and photogrammetric surveys. Their uses in hydrographic surveying are very similar to topographic surveying. Vertical controls are necessary in obtaining data required for waterfront structures, dredging, flood control, and the like. In photogrammetric surveying, vertical controls are necessary for the preparation of controlled mosaic and the preparation of relief maps from aerial photographs.

REPRESENTATION OF RELIEF

Before the procedure for locating details from a station is described, the subject of "topographic relief" should be explained. Relief is the term applied to variance in the vertical configuration of the earth's surface. You have seen how relief can be shown in a plotted profile or cross-section. These, however, are views on a vertical plane, but a topographic map is a view on a horizontal plane. On such a map, relief may be indicated by the following methods.

A "relief model" is a three-dimensional relief presentation—a molded or sculptured model, to suitable horizontal and vertical scales, of the hills and valleys in the area.

"Shading" is a pictorial method of showing relief, in which light and dark areas are used to suggest the shadows which would be created by parallel rays of light, shining across the area at a given angle.

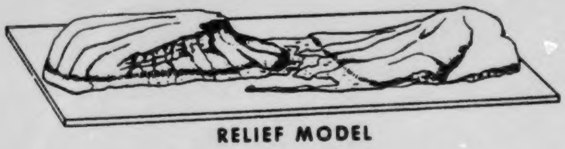
"Hachures" is a pictorial method similar to shading, except that the light-and-dark pattern is created by short "hachure lines," drawn parallel to the steepest slopes. Relative steepness or flatness is suggested by varying the lengths and weights of the lines.

"Contour lines" are lines of equal elevation—that is, each contour line on a map is drawn through a succession of points, all of which are at the same elevation. A "contour" is the real-life equivalent—that is, a line of equal elevation on the earth's surface.

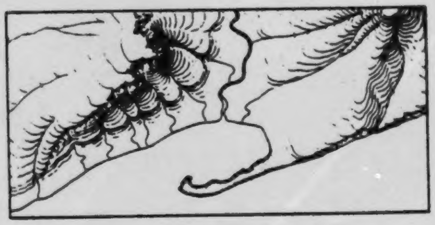
All of these methods of indicating relief are illustrated in figure 15-2. The contour-line method is the one most commonly used on topographic maps.

CONTOUR LINES

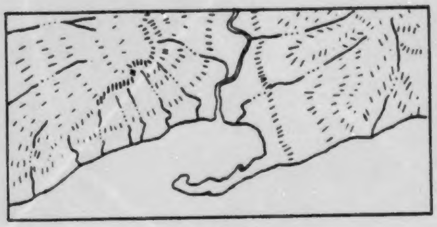
As stated in chapter 8, a CONTOUR is an imaginary level line on the ground surface, and the corresponding line on a corresponding map



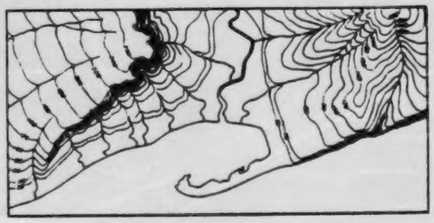
RELIEF MODEL



SHADED FORM LINES



HACHURES



CONTOURS

45.405

Figure 15-2.—Methods of indicating relief.

of an area is called a CONTOUR LINE. The best illustration of a contour is the line traced by the surface of a still body of water in a pond or a lake. The intersection of the water with the ground is called the CONTOUR and when this line is drawn on a map, the term used will be CONTOUR LINE.

Contour lines indicate a vertical distance above or below a datum plane. Starting at sea level, normally the zero contour, each contour line represents an elevation above sea level. The vertical distance between adjacent contour lines is known as the contour interval and the amount of the contour interval is given in the marginal information. On most maps, the contour lines are printed in brown. Starting at zero elevation, every fifth contour line is drawn with a heavier line. These are known as INDEX CONTOURS and some place along each index contour, the line is broken and its elevation is given. The contour lines falling between index contours are called INTERMEDIATE CONTOURS. They are drawn with a finer line than the index contours and usually do not have their elevations given.

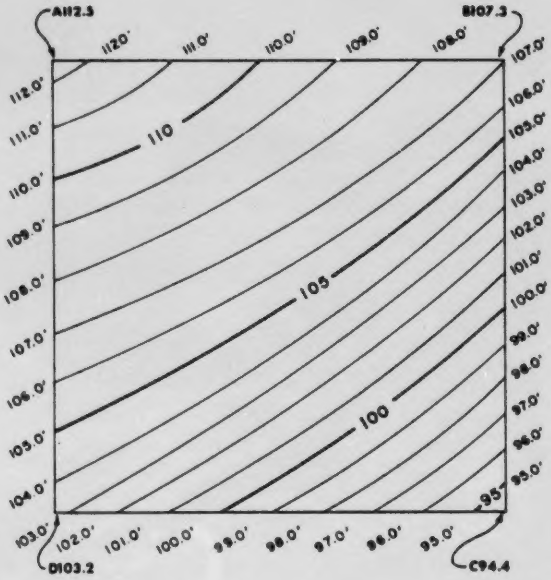
GROUND POINT SYSTEMS

The essential data for showing relief by contour lines consists of the elevation of a sufficient number of "ground points" in the area. Methods of determining the horizontal and vertical locations of these ground points are called "ground point systems." The systems most frequently used are (1) tracing contours, (2) grid, (3) control points, and (4) cross profiles. In practice, combinations of these methods may be used in one survey.

Tracing Contours

In the tracing contours system, the ground points located are points on the actual contours. Points on a given contour are plotted on the map, and the contour line is drawn through the plotted points. The method may be illustrated by the following simple example.

Assume that the traverse shown in figure 15-3 runs around the perimeter of a small field. The elevations at corners A, B, C, and D are as indicated. It is abvious that the ground slopes downward from AB toward DC, and from AD toward BC. You want to locate contours at a "contour interval" of 1 ft—that is, you want to plot the 112-ft contour line, the 111-ft contour line, the 110-ft contour line, and so on.



45.406
Figure 15-3.—Traverse with contour lines.

You stand at station A with a hand level. The elevation of this station is 112.5 ft. Assume that the vertical distance from your eye level to the ground is 5.7 ft. Then with the hand level at your eye, and with you standing on station A, the H.I. is 112.5 + 5.7, or 118.2 ft.

If a level rod is set up anywhere on the 112.0-ft contour, the reading you would get from station A would be 118.2 - 112.0, or 6.2 ft. Therefore, to determine the point where the 112.0-ft contour crosses AB, all you need to do is have the rodman back out from A along AB until he comes to the point where you read 6.2 ft on the rod. The point where the 112.0-ft contour crosses AD can be determined in the same manner. The distance from A to each point can be measured, and the distance from A to the 112.0-ft contour on AB and AD then recorded.

When all of the contours have been located on AB and AC, you can shift to station C and carry out the same procedure to locate the contours along BC and CD. You have now located all the points where contours at a 1-ft interval intersect the traverse lines. If the slope of the ground is uniform (as it is presumed to be in figure 15-3), you can plot the contour lines by simply drawing lines between points of equal elevation, as shown in that figure. If there were irregularities in the slope, you would send the rodman out along one or more lines laid across the irregular ground, locating the contours on these lines just as you located them on the traverse lines.

Grid Coordinate System

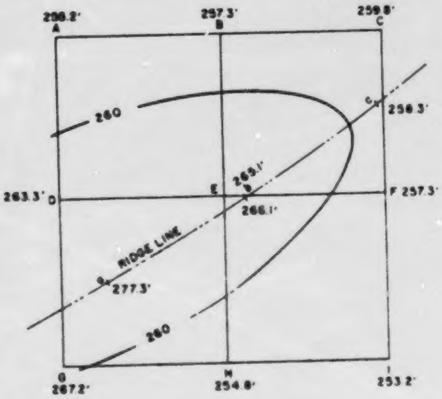
In the grid coordinate system, the area is laid out in squares of convenient size, and the elevation of each corner point is determined. While this method lends itself to use on relatively regular ground, ridge or valley lines must be located by "spot" elevations taken along the lines. The locations of the desired contours are then determined, on the ridge and valley lines and on the sides of the squares, by "interpolation." This gives a series of points, through which the contour lines may be drawn.

Figure 15-4 illustrates this method. Assume that the squares here measure 200.0 ft on a side. Points a, b, and c are points on a ridge line, also 200.0 ft apart. It is desired to locate and draw the 260.0-ft contour line. By inspection, you can see that the 260.0-ft contour must cross AD (elevation of A 255.2 ft, elevation of D 263.3 ft). At what point does the 260.0-ft contour cross AD? This can be determined by proportional equation as follows.

Assume that the slope from A to D is uniform. The difference in elevation is 8.1 ft (263.3 - 255.2) for 200.0 ft. The difference in elevation between 255.2 ft and 260.0 ft (elevation of the desired contour) is 4.8 ft. The distance from A to the point where the 260.0-ft contour crosses AD is the value of x in the proportional equation,

$$\frac{8.1}{200} = \frac{4.8}{x},$$

or 118.5 ft. Lay off 118.5 ft from A on AD and make a mark.



45.407
Figure 15-4.—Grid system of ground points.

The points where the 260.0-ft contour crosses BE, EF, EH, and GH can be located and marked in the same manner. The 260.0-ft contour crosses the ridge, obviously, between point b (elevation 266.1 ft) and point c (elevation 258.3 ft). The distance between b and c is again 200.0 ft; the location of the point of crossing can be obtained by the same procedure just described.

You now have six plotted points: one on the ridge line between b and c, and the others on AD, BE, EF, EH, and GH. A line sketched by hand through these points is the 260.0-ft contour line. Note that the line is, in effect, the line that would be formed by a horizontal plane, passed through the ridge at an elevation of 260.0 ft. Note, too, that a contour line changes direction at a ridge summit.

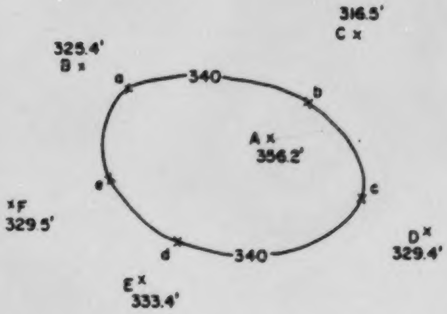
Control Points

The preceding explanation illustrates the fact that any contour line may be located on a uniform slope, between two points of known elevation a known distance apart, by interpolation. It also demonstrates how a ridge line is located by spot elevations; a valley line would be located in the same manner.

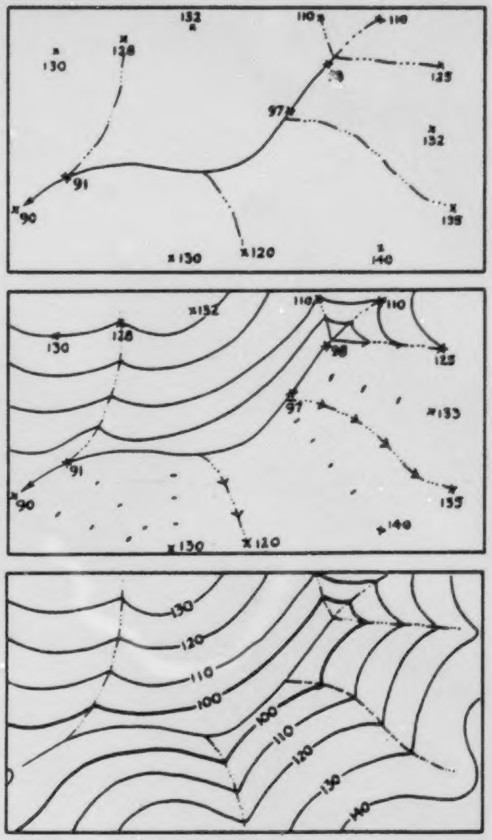
If all the important irregularities in an area (ridges, valleys, and any other points where elevation changes radically) are located and plotted, a contour map of the area can be drawn by interpolating the desired contours between the control points.

A very elementary application of the method is shown in figure 15-5. Point A is the summit of a more or less conical hill. A spot elevation is taken here. Points B, C, D, E, and F are points at the foot of the hill; spot elevations are also taken here. It is desired to draw the 340.0-ft contour. Point a on the contour is interpolated on the line from A to B; point b is interpolated on the line from A to C; point c is interpolated on the line from A to D; and so on.

Figure 15-6 shows a more complicated example in which contours are interpolated and



45.408
Figure 15-5.—Control point method of locating contour.

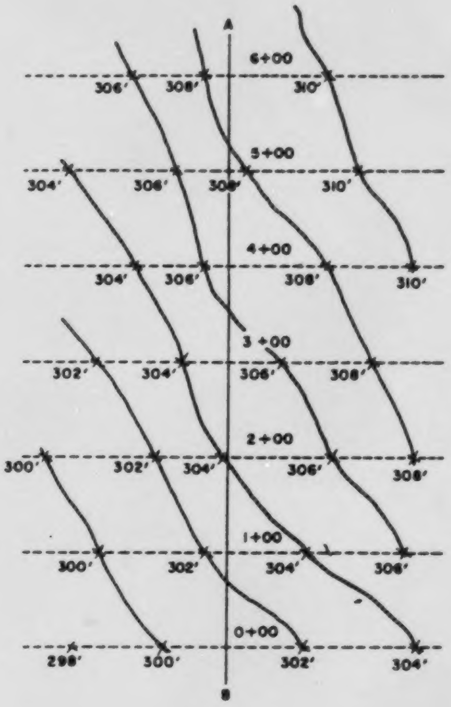


45.409
Figure 15-6.—Sketching contours by interpolation between control points of known elevation.

sketched between controlling spot elevations taken along a stream.

Cross Profiles

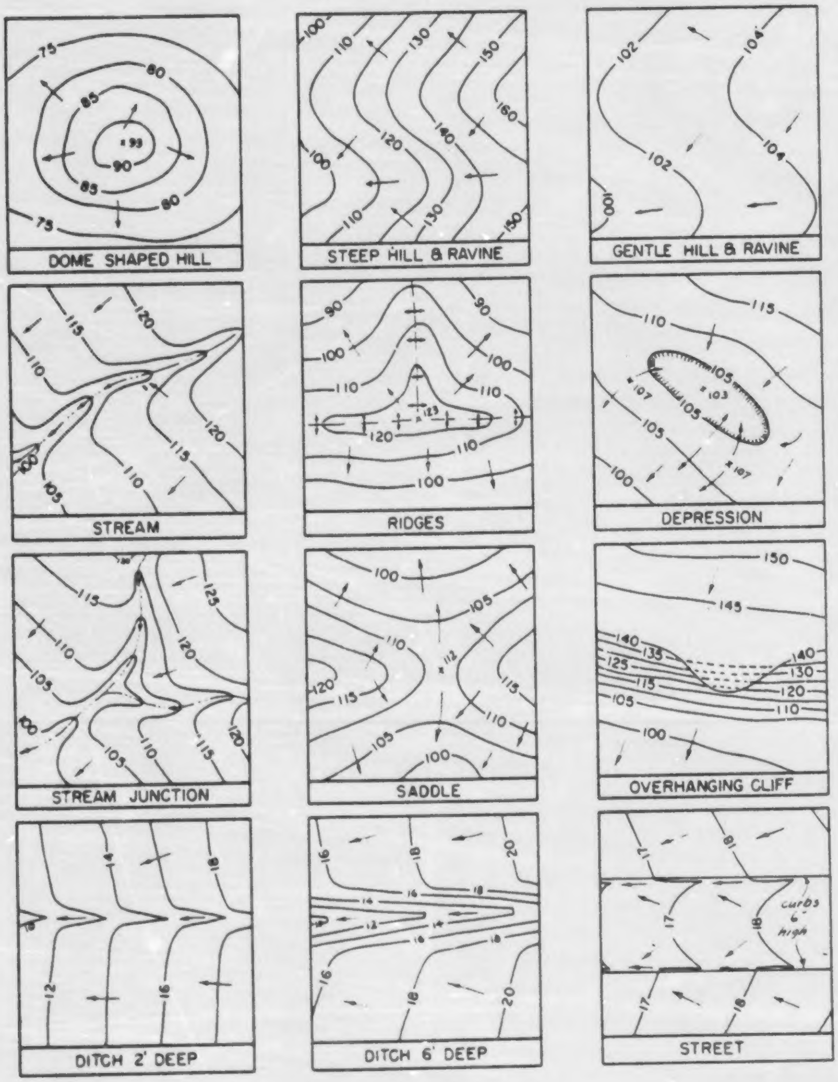
In the cross profile system, elevations are taken along selected lines which are at right angles to a traverse line. Shots are taken at regular intervals and/or at breaks in the ground slope. The method is illustrated in figure 15-7. The line AB is a traverse, along which 100-ft stations are shown. On each of the dotted cross-section lines, contours are located. The particular contour located at a particular station depend on (1) the ground elevations, and (2) the prescribed contour interval, which in this instance is 2 ft. The method used to locate the contours is the one previously described for tracing contour system. When the even-numbered, 2-ft interval contours are located on all the cross-profile lines, the contour lines are drawn through the points of equal elevation.



45.410
Figure 15-7.—Cross profiles.

CHARACTERISTICS OF CONTOUR LINES

A contour line is a line of equal elevation; therefore, two different lines must indicate two different elevations. It follows that two different contour lines cannot intersect or otherwise contact each other, except at a point where a vertical or overhanging surface (such as a vertical or overhanging face of a cliff) exists on the ground. In figure 15-8, "overhanging cliff," you can see how the segments of contour lines on an overhanging cliff are made dotted or "hidden" lines. Aside from the exception mentioned, a point where two different contour lines intersected would be a point with two different elevations—an obvious impossibility.



45.411
Figure 15-8. — Typical contour formations.

In forming a mental image of the surface configuration from a study of contour lines, it is helpful for you to remember that a contour line is a LEVEL line; that is, a line which would be formed by a horizontal plane passed through the earth at the indicated elevation. If you keep this concept of "leveliness" in mind, you can usually get the "feel" of the rise and fall of the ground as you study the contour lines on the map.

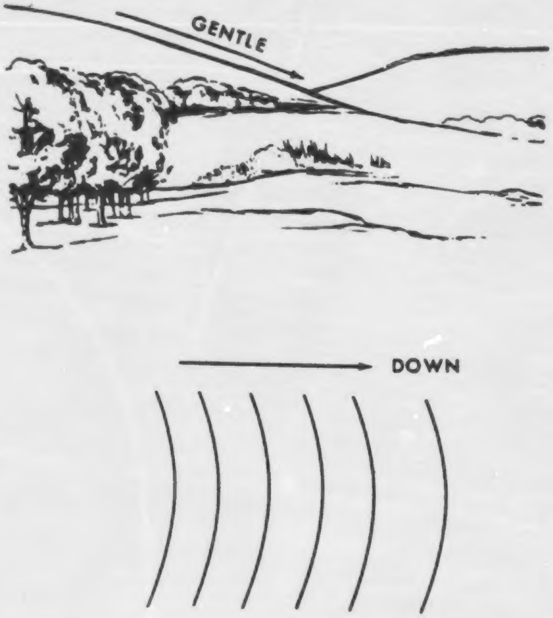
A contour line must close on itself somewhere — either within or beyond the boundaries of the map. A line which appears on the map completely closed may indicate either a summit or a depression. If the line indicates a depression, this fact is sometimes shown by a succession of short hachure lines, drawn perpendicular to the "inner" side of the line, as shown in figure 15-8, "depression." A contour line marked in this fashion is called a "depression contour."

On a horizontal or level plane surface, the elevation of all points on the surface is the same. Therefore, since different contour lines indicate different elevations, there can be no contour lines on such a surface. On an inclined plane surface, contour lines at a given equal interval will be straight, parallel to each other, and equidistant.

A number of typical contour formations are shown in figure 15-8. For purposes of simplification, horizontal scales are not shown; however, you can see that various intervals are represented. The arrows shown indicate the direction of flow of water.

Generally, the spacing of the contour lines indicates the nature of the slope. Contour lines evenly spaced and wide apart indicate a uniform, gentle slope. (See fig. 15-9.) Contour lines evenly spaced and close together indicate a uniform, steep slope. The closer the contour lines to each other, the steeper the slope. (See fig. 15-10.) Contour lines closely spaced at the top and widely spaced at the bottom indicate a concave slope (fig. 15-11). Contour lines widely spaced at the top and closely spaced at the bottom indicate a convex slope (fig. 15-12).

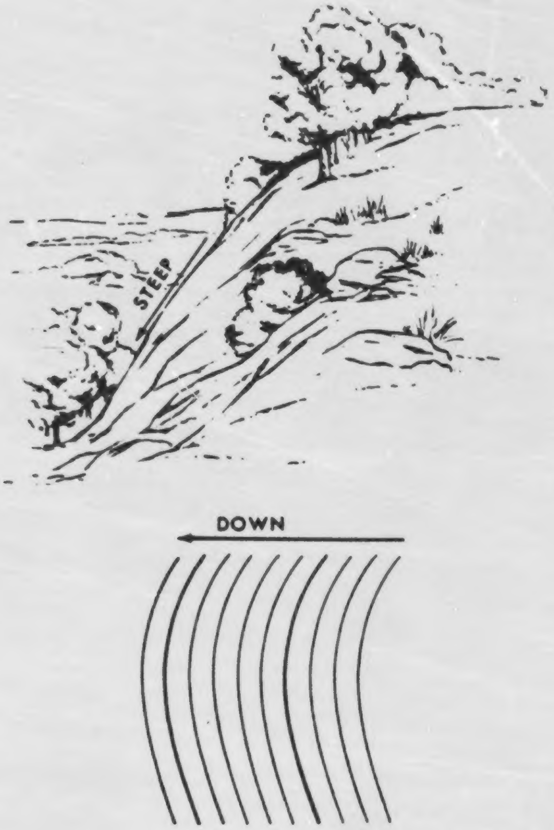
In order to show the relationship of land formations to each other and how they would be symbolized on a contour map, stylized PANORAMIC SKETCHES of the major relief formations were drawn and the contour map of each sketch developed. A panoramic sketch is a pictorial representation of the terrain in elevation and perspective as seen from one point of observation. It shows the horizon which is always of military importance, and intervening features such as crests, woods, structures, roads, fences, and so on. Each of figures 15-13 through 15-19 shows a sketch and map with a different relief feature and its characteristic contour pattern. Each of the relief features illustrated are defined in the following paragraphs.



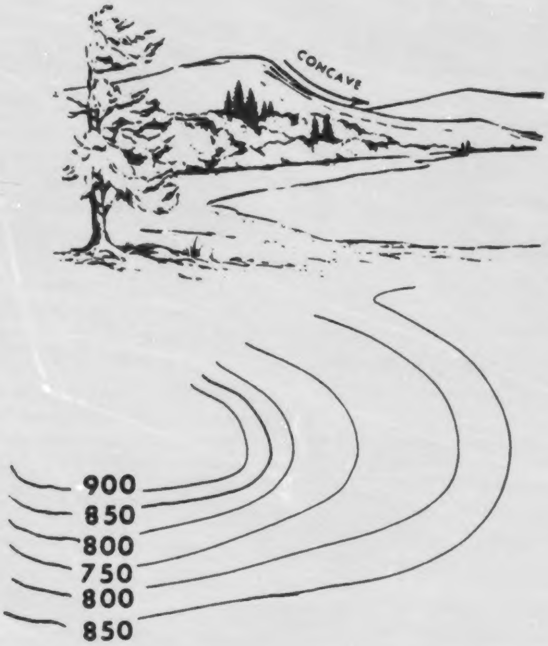
45.784
Figure 15-9. — Uniform gentle slope.

HILL. A point or small area of high ground (fig. 15-13). When you are located on a hilltop, the ground slopes down in all directions.

VALLEY. A stream course which has at least a limited extent of reasonably level ground bordered on the sides by higher ground (fig. 15-14). The valley generally has maneuver room within its confines. Contours indicating a valley are U-shaped and tend to parallel a major stream before crossing it. The more gradual the fall of a stream, the farther each contour parallels it. The CURVE of the contour crossing ALWAYS points upstream.



45.785
Figure 15-10.—Uniform steep slope.



45.786
Figure 15-11.—Concave slope.

DRAW. A less-developed stream course in which there is essentially no level ground and, therefore, little or no maneuver room within its sides and towards the head of the draw. Draws occur frequently along the sides of ridges, at right angles to the valley between them. Contours indicating a draw are V-shaped, with the point of the "V" toward the head of the draw.

RIDGE. A line of high ground, with normally minor variations along its crest (fig. 15-15). The ridge is not simply a line of hills; all points of the ridge crest are appreciably higher than the ground on both sides of the ridge.

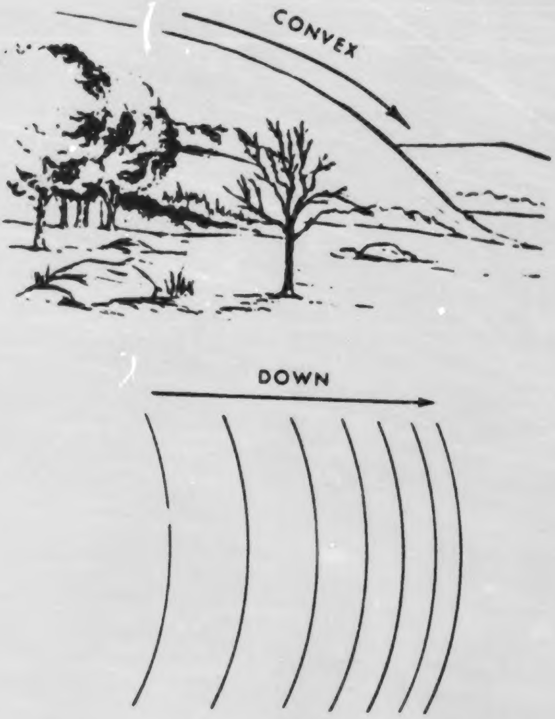
SPUR. A usually short, continuously sloping line of higher ground normally jutting out from the side of a ridge (fig. 15-15). A spur is often formed by two roughly parallel streams cutting draws down the side of the ridge.

SADDLE. A dip or low point along the crest of a ridge. A saddle is not necessarily the lower ground between the two hilltops; it may be simply a dip or break along an otherwise level ridge crest (fig. 15-16).

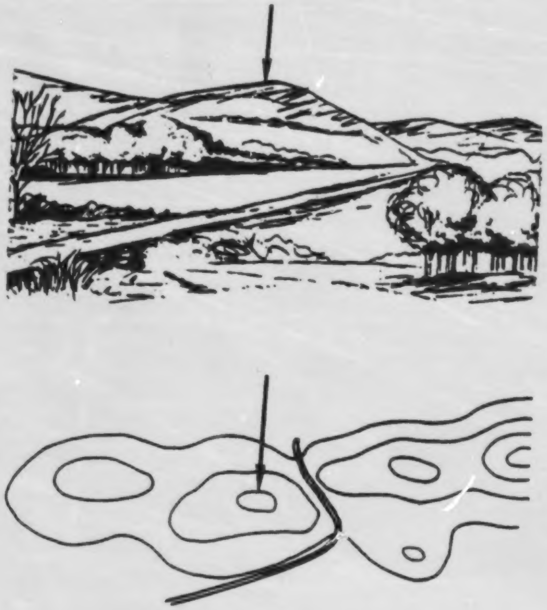
DEPRESSION. A low point or sinkhole, surrounded on all sides by higher ground (fig. 15-17).

CUTS AND FILLS. Man-made features by which the bed of a road or railroad is graded or leveled off by cutting through high areas and filling in low areas (fig. 15-18) along the right-of-way.

CLIFF. A vertical or near vertical slope (fig. 15-19). When a slope is so steep that it cannot be coalescing, it is shown by a ticked "carrying" contour or contours. The ticks always point toward lower ground.



45.787
Figure 15-12.—Convex slope.



45.788
Figure 15-13.—Hill.

MAP SCALES AND CONTOUR INTERVALS

A topographic map is called large scale, intermediate scale, or small scale in accordance with the following criteria:

Large scale: 1 in. = 100 ft or less
Intermediate scale: from 1 in. = 100 ft to 1 in. = 1000 ft
Small scale: 1 in. = 1000 ft or more.

The designated contour interval varies according to the purpose and scale of the map and the character of the terrain. Table 15-1 shows the recommended contour intervals that may be used when preparing a topographic map.

CONTOUR MAP CONSTRUCTION

If an EA can perform ordinary engineering drafting chores, he will not have any difficulty in constructing a topographic map. Up to some degree, contour lines must be drawn by estimation. His knowledge of contour line characteristics and the configuration of the terrain they represent will be a great help. He must use his skill and judgment so that the contour lines he draws may best represent the actual configuration of the ground surface.

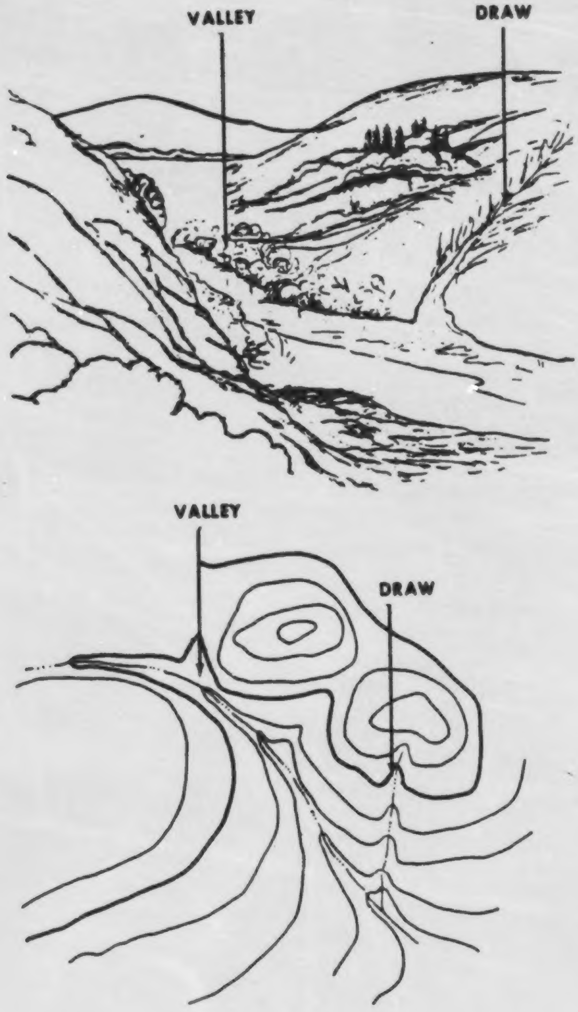
Basically, the construction of a contour map consists of three operations. They are:

1. Plotting of horizontal control which will serve as the framework of the map.

2. Plotting of details including the map location of points of known ground elevation, simply called GROUND POINTS. These ground points or contour points will be used as guides for the proper location of the contour lines.

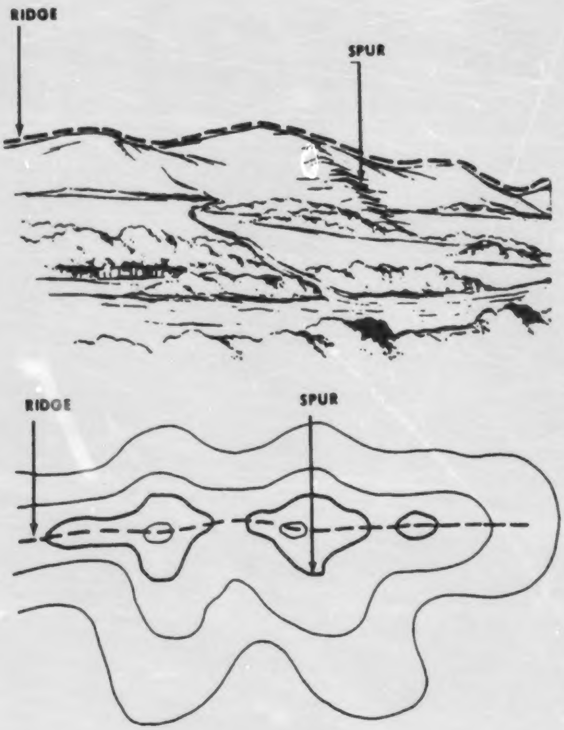
3. Construction of contour lines at given contour intervals.

Special care is taken in the field to locate RIDGE LINES and VALLEY LINES, for these



45.789
Figure 15-14.—Valley and draw.

lines are usually drawn first on the map before actual contour points are plotted. (See view A, fig. 15-20.) As contours ordinarily change direction sharply where they cross these lines, and since the slopes of ridges and valleys are fairly uniform, they are aids in drawing the correct contour lines. After the ridge and valley lines are plotted, contour crossings are spaced (by interpolation) along them before any attempt is made to interpolate or to draw the complete contour lines. (See view B, fig. 15-20.)

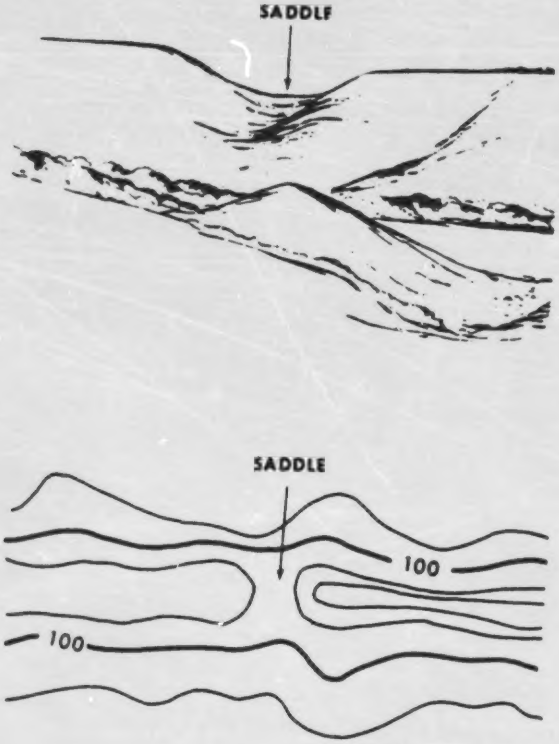


45.790
Figure 15-15.—Ridge and spur.

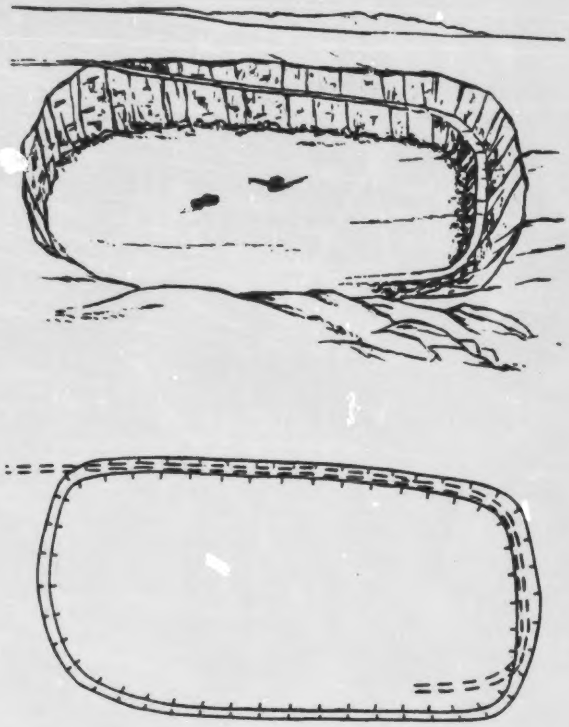
Contour lines are smoothly drawn freehand with uniform width and with best results if a contour pen is used. Breaks in the lines are provided to leave spaces for the elevations. The numbers which represent these elevations are written in such a way that they may be read from one or two sides of the map; some authorities prefer that they are also written in such a way that their tops are towards the uphill. Spot elevations are shown at important points, such as road intersections.

View C, figure 15-20, illustrates the completed contour map. For more refined work, the map must be traced, using a contour pen, on a tracing cloth or tracing paper to allow reproduction of more copies, if necessary.

If the nature of terrain and weather conditions permit, a topographic map is best accomplished with the use of a plane table. In this way, the fieldwork and sketching are all done



45.791
Figure 15-16.—Saddle.



45.792
Figure 15-17.—Depression.

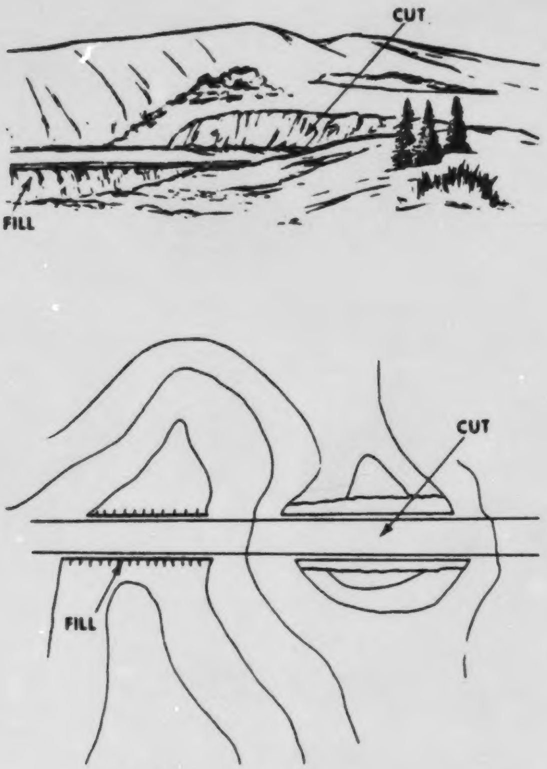
in the field. Detail positions are plotted to scale with respect to the plotted position of the occupied station. They are normally located by radiating rays and distances measured by stadia, and plotted directly on the plane table sheet. The ground elevation at each point is determined and shown on the sheet at the plotted position. On a large scale map, it is often possible to represent the true shape of features to scale. On small scale maps, buildings and other features must often be symbolized, with the symbol centered on the true position, but drawn larger than the scale of the map. Such detail is portrayed on the map by means of standardized topographic symbols; these symbols are shown in figure 15-21.

After a number of key points have been located (usually from one occupied station) and plotted, sketching of the contour lines is started.

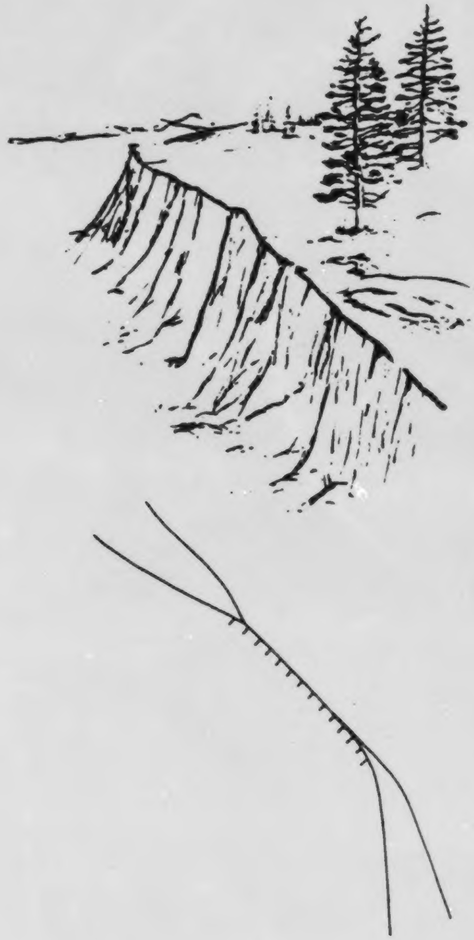
Contour lines are drawn on the map by logical contouring. Ground elevations are determined at key points plotted on the plane table sheet. Such key points are located at the following positions:

1. Hill or mountain tops
2. Ridge lines.
3. Top and foot of steep slopes.
4. Valleys and streams.
5. Saddles between hills.

Views A to C, figure 15-20, also represent the evolution of a topographic map using the plane table. Key points with description and elevation are shown as in view A. To aid in drawing the contours, the drainage and the ridge lines are sketched. The contours crossing these lines are marked by interpolation. Along the slopes, the contours are interpolated between the marked key points using short dashes shown



45.793
Figure 15-18.—Contour (cut and fill).



45.794
Figure 15-19.—Cliff.

in view B. The final step is to join the equivalent contour lines as in view C.

When joining contour lines, the topographer must utilize his view of terrain. If the terrain along the slopes is curved smoothly, the topographer should join his contours with smooth convex or concave curves to depict the terrain. Relatively straight and angular slopes are reflected by more or less straight contour lines. Slopes with numerous small drains should be contoured using lines with indentations at the proper locations to show this condition. This method of contouring should continue as the survey progresses. In addition to the sketching, the topographer should constantly be alert for any additional key points around him that may be used for future contouring. These points are included in the survey and marked to be used at some future setup.

When contouring, it must be remembered that stream and ridge lines have a primary influence on the direction of the contour lines, and that the slope of the terrain controls the spacing of the contour lines. Contour lines crossing a stream follow the general direction of the stream on both sides, then cross the stream in a fairly sharp V that points upstream. Also, contour lines curve around the nose of ridges in the form of a U pointing downhill and cross ridge lines at approximately right angles.

330

45.796

Table 15-1.—Recommended Contour Intervals—Topographic Map

TYPES OF TOPOGRAPHIC MAP	NATURE OF TERRAIN	RECOMMENDED CONTOUR INTERVAL IN FT.
LARGE SCALE	Flat	0.5 or 1
	Rolling	1 or 2
	Hilly	2 or 5
INTERMEDIATE SCALE	Flat	1, 2, or 5
	Rolling	2 or 5
	Hilly	5 or 10
SMALL SCALE	Flat	2, 5, or 10
	Rolling	10 or 20
	Hilly	20 or 50
	Mountainous	50, 100, or 200

INTERPOLATING CONTOUR LINES

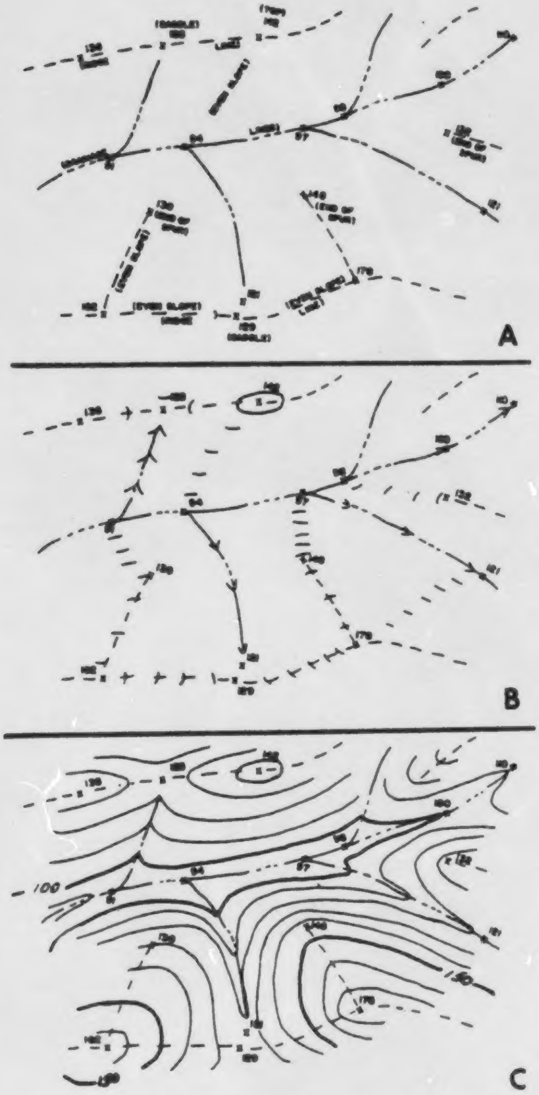
In the examples of interpolation previously given, a single contour line was interpolated between two points of known elevation, a known horizontal distance apart, by mathematical computation. In actual practice, it is usually necessary to interpolate more than one line between a pair of points, and large numbers of interpolations between many pairs of points are required. Mathematical computation for the location of each line would be time consuming, and would be used only in a situation in which contour lines had to be located with an unusually high degree of accuracy.

For most ordinary contour-line drawing, one of several rapid methods of interpolation is used. In each case, it is assumed that the slope between the two points of known elevation is uniform.

Figure 15-22 illustrates the use of an engineer's scale to interpolate the contours at a 2-ft interval between A and B. The difference in elevation between A and B is between 11 and 12 ft. Select the scale on the engineer's scale which has 12 graduations for a distance which comes close to matching that between A and B on the map. In figure 15-22, this is the 20 scale. Let the 0 mark on the 20 scale represent 530.0 ft. Then the 0.2 mark on the scale will represent 530.2 ft, which is the elevation of A. Place this mark on A, as shown.

If the 0 mark on the scale represents 530.0 ft, then the 11.7 mark must represent 530.0 + 11.7, or 541.7 ft, which is the elevation of B. Place the scale at a convenient angle to the line from A to B, as shown, and draw a line from the 11.7 mark to B. You can now project the desired contour line locations from the scale to the line from A to B by drawing lines from the appropriate scale graduations (2, 4, 6, and so on) parallel to the line from the 11.7 mark to B.

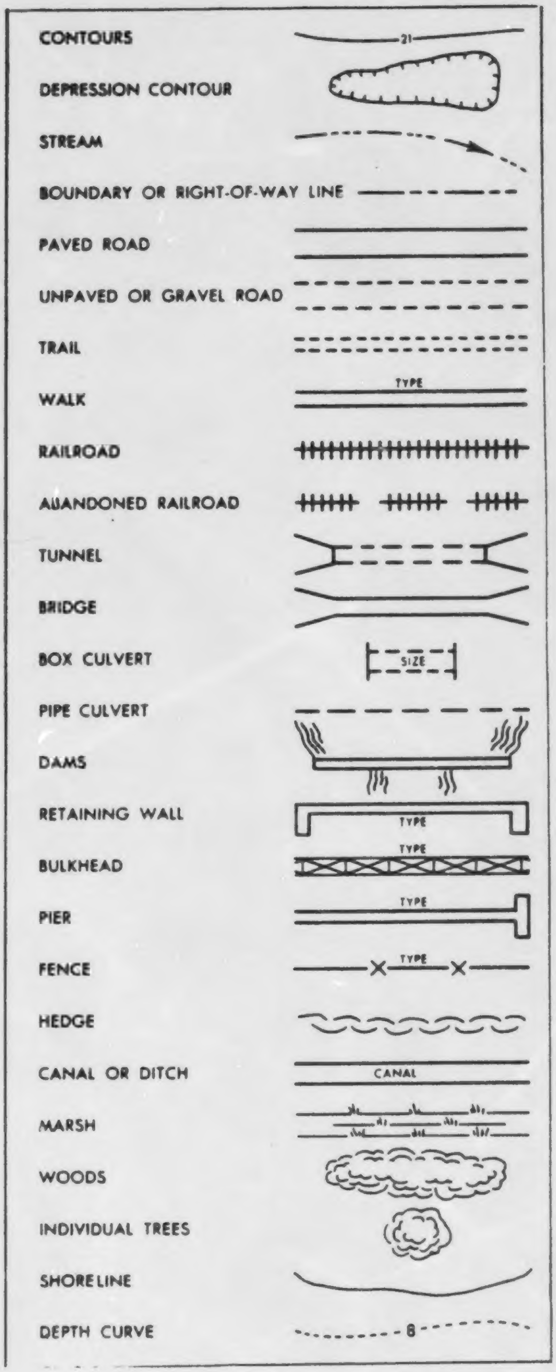
Figure 15-23 illustrates a graphic method of interpolating contour lines. On a transparent sheet (tracing cloth is excellent for the purpose), draw a succession of equidistant parallel lines.



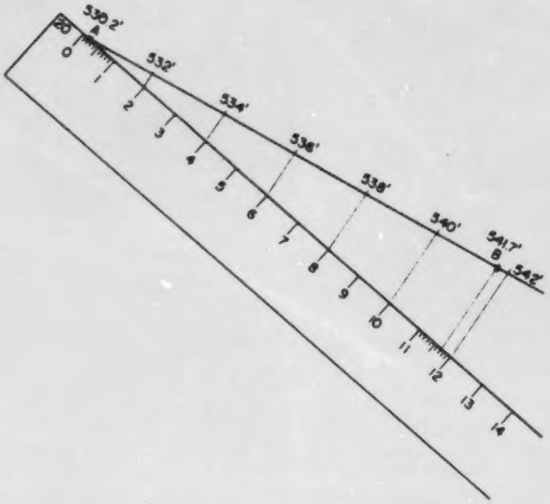
45.795
Figure 15-20.— Plotting detail and contouring.

Number the lines as shown in the left margin, the 10th line being number 1, the 20th number 2, and so on. The interval between each pair of adjacent lines, then, represents 0.1 ft.

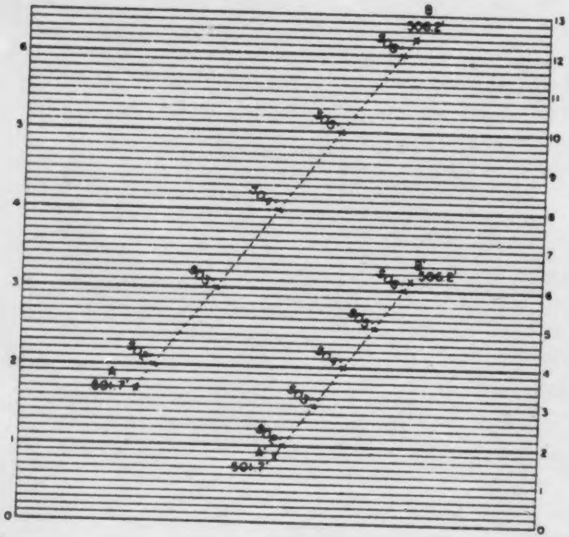
Figure 15-23 shows how you can use this sheet to interpolate contour lines at a 1-ft interval between point A and point B. Place



45.414.0
Figure 15-21.—Commonly used map symbols.



45.412
Figure 15-22.—Interpolating contour lines with a scale.



45.413
Figure 15-23.—Graphic method of interpolating contour lines.

the sheet on the map so that the line which represents 1.7 ft (elevation of A is 500.0 + 1.7, or 501.7 ft) is on A, and so that the line which represents 6.2 ft (elevation of B is 500.0 + 6.2, or 506.2 ft) is on B. You can see how you can then locate the 1-ft contours between A and B.

For a steeper slope the contour lines would be closer together. If they were considerably closer, you might find it advisable to give the lines on the lined sheet different values, as indicated by the numerals in the right-hand margin. Here the space between each pair of lines represents, not 0.1 ft, but 0.2 ft. Points A' and B' have the same elevations as points A and B, but the fact that the horizontal distance between them is much shorter shows that the slope between them is much steeper. You can see how the 1-ft contours between A' and B' can be located, using the line values shown in the right margin.

Still a third method of rapid interpolation involves the use of a rubber band, marked off in appropriate equal decimal intervals. The band is stretched so as to bring the appropriate graduations on the points.

LOCATING TOPOGRAPHIC DETAILS

When the stations (or "control points," or "instrument points") have been horizontally and vertically located, the next major step in a topographic survey is to locate the details in the vicinity of each station. These details consist of (1) all features, either natural or artificial (man-made), which will appear on the map, and (2) enough ground points and spot elevations to make the drawing of contour lines possible.

You could locate details horizontally by transit-tape and vertically by differential leveling; but this would be a time-consuming process requiring the chaining of many distances and the taking of many levels. Consequently, detail location is usually done either by transit-stadia or by the still more rapid plane-table procedure.

LOCATING DETAILS BY TRANSIT-TAPE

The same procedure used for tying in, as explained earlier in previous chapters, is used for locating details with the transit and tape. The detail is located by directions and distances or a combination of both. A method or a combination of methods which requires the least time in a particular situation must be used.

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The dimensions of structures such as buildings are measured directly with tapes. If details are numerous, each one may be assigned a number in the sketch and keyed to a legend of some sort to avoid overcrowding, and azimuths are used for directions instead of deflection angles to minimize confusion. The following methods, which are self-explanatory, are commonly used for locating details.

1. By angle and distance from transit stations.
2. By angles from two transit stations.
3. By distances from two known points.
4. By an angle from one station and distance from another.
5. By swing offsets and range ties.

LOCATING DETAILS BY TRANSIT-STADIA

The method of determining horizontal distance and elevation by stadia is explained in chapters 11 and 14. Figure 15-24 shows a field note for locating topographic details by transit-stadia. The details shown by number in the sketch on the "remarks" side are listed by number in the column headed "Obj." on the data side. On the data side you see, first, that the instrument station was station D_1. Below this the entry "BS A, angles rt." means that the direction of each detail point from D_1 is given in terms of the horizontal angle, measured to the right, between the line from D_1 to A and the line from D_1 to the detail point.

For example: to determine the direction of point 1, the transit telescope was trained on A and the zeros were matched. The telescope was then turned right to train on point 1, and the horizontal angle (30° 10') was read.

In the third line from the top on the data side you see "El. 532.4, H.I. 4.8." This means that the elevation of D_1 is 532.4 ft, and the vertical distance between D_1 and the line of sight through the telescope is 4.8 ft.

For the horizontal distance and elevation of each point, a level rod was set up on the point, and the transit telescope was trained on the 4.8 graduation on the rod. With the telescope thus trained, the vertical angle between D_1 and the observed point, as indicated by the horizontal crosshair, could be read on the vertical circle. At the same time, the rod intercept, as indicated by the stadia hairs, was read. The rod intercept was entered in the column headed "Rod Int.," and the vertical angle in the next column, with each vertical angle marked minus or plus as appropriate.

From the rod intercept and the vertical angle, the horizontal distance (entered in the fifth column) and difference in elevation (sixth column) were determined from a "stadia reduction table." Figure 15-25 shows the page from such a table which applies to the data for point 1 in figure 15-24. For this point the vertical angle is - 3° 26', and the rod intercept is 6.23 ft. In the table, under 3° and opposite 26', you see that the multiplier for horizontal distance is 99.64, while that for difference in elevation is 5.98. Ignoring focal distance, the horizontal distance is 6.23 x 99.64, or (to the nearest foot) 621 ft. The difference in elevation is 6.23 x 5.98, or - 37.3 ft. To these the corrections for focal distance, given at the bottom of the page, should be added. For an instrument with a focal distance of 1 ft you add 1 ft to the horizontal distance (making a total horizontal distance of 622 ft) and 0.06 ft to the difference in elevation, which makes that difference round off at - 37.4 ft.

In the first column on the "remarks" side the elevation of each point is entered, computed as follows. For point 1 the elevation is the elevation of the instrument station D_1 (which is 532.4 ft) minus the difference in elevation (37.4 ft), or 495.0 ft. You subtracted the difference in elevation in this case because the vertical angle you read for point 1 was negative. For a positive vertical angle (as in the cases of points 12 through 17) you add the difference in elevation.

If you knew the azimuth of D_1A, you could indicate your directions in azimuths instead of in angles right from D_1A. Suppose, for example, that the azimuth of D_1A was 26° 10'. You would train the telescope on A and set the horizontal limb to read 26° 10'. Then when you trained on any detail point, you would read the azimuth of the line from D_1 to the detail point on the horizontal limb.

LOCATING DETAILS BY PLANE TABLE

Before the method of locating details by plane table is described, some further information about the plane table alidade (which was briefly described in chapter 9) must be mentioned. The telescope on an alidade is similar in most respects to that on a transit. It has a telescope level vial and vertical motion tangent screw for bringing the telescope to an exact

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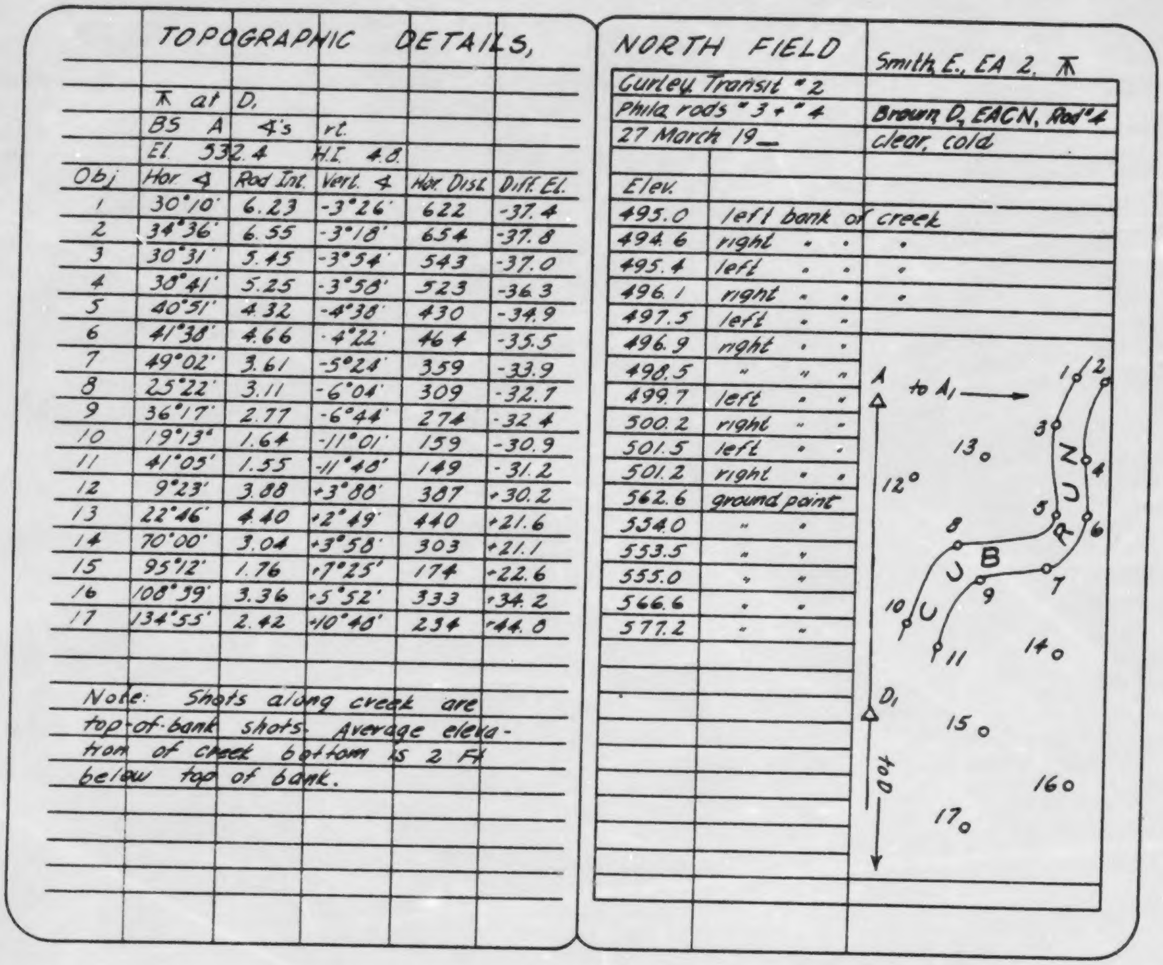


	TOPOGRAPHIC DETAILS,				
	⊼ at	D,			
	BS A	∢'s	rt.		
	El. 532.4		H.I. 4.8		
Obj	Hor ∢	Rod Int.	Vert. ∢	Hor Dist.	Diff. El.
1	30°10'	6.23	-3°26'	622	-37.4
2	34°36'	6.55	-3°18'	654	-37.8
3	30°31'	5.45	-3°54'	543	-37.0
4	38°41'	5.25	-3°58'	523	-36.3
5	40°51'	4.32	-4°38'	430	-34.9
6	41°38'	4.66	-4°22'	464	-35.5
7	49°02'	3.61	-5°24'	359	-33.9
8	25°22'	3.11	-6°04'	309	-32.7
9	36°17'	2.77	-6°44'	274	-32.4
10	19°13'	1.64	-11°01'	159	-30.9
11	41°05'	1.55	-11°48'	149	-31.2
12	9°23'	3.88	+3°88'	387	+30.2
13	22°46'	4.40	+2°49'	440	+21.6
14	70°00'	3.04	+3°58'	303	+21.1
15	95°12'	1.76	+7°25'	174	+22.6
16	108°39'	3.36	+5°52'	333	+34.2
17	134°55'	2.42	+10°40'	234	+44.0

Note: Shots along creek are top-of-bank shots. Average elevation of creek bottom is 2 Ft below top of bank.

NORTH FIELD		Smith E., EA 2, ⊼
Gurley Transit #2		
Phila rods #3 + #4		Brown D, EACN, Rod #4
27 March 19__		clear, cold
Elev.		
495.0	left bank of creek	
494.6	right " "	"
495.4	left " "	"
496.1	right " "	"
497.5	left " "	
496.9	right " "	
498.5	" " "	
499.7	left " "	
500.2	right " "	
501.5	left " "	
501.2	right " "	
562.6	ground point	
534.0	" "	
553.5	" "	
555.0	" "	
566.6	" "	
577.2	" "	

45.7.14
Figure 15-24. — Notes for locating topographical details by transit-stadia.

horizontal. However, the mechanism for measuring vertical angles is different from that on a transit.

With plane table equipment you can plot details directly on the map in the field. Some of the advantages over the transit-stadia method are (1) the map is made while you are surveying right at the area, (2) observed irregular lines can be sketched in, (3) field notes and their reduction are unnecessary, and (4) much less time is required to produce a map. Some disadvantages are (1) more time in the field is required, (2) bad weather will hold everything up, though the same weather might not hold up a transit-stadia crew, (3) control must be plotted in advance for precise work, (4) distance must be scaled if a length is to be taken from the map, (5) many more items, some of them awkward to handle, must be transported, and (6) it takes considerably more time for a man to become proficient with the plane table.

The procedure for plane-table plotting of the details shown in figure 15-24 is roughly as follows. You take into the field a sheet of plane table paper of suitable size, on which the control traverse (the line through D_1 to A) is already plotted to suitable scale. Naturally

Minutes	0°		1°		2°		3°	
	Hor. dist.	Diff. elev.	Hor. dist.	Diff. elev.	Hor. dist.	Diff. elev.	Hor. dist.	Diff. elev.
0........	100.00	0.00	99.97	1.74	99.88	3.49	99.73	5.23
2........	100.00	0.06	99.97	1.80	99.87	3.55	99.72	5.28
4........	100.00	0.12	99.97	1.86	99.87	3.60	99.71	5.34
6........	100.00	0.17	99.96	1.92	99.87	3.66	99.71	5.40
8........	100.00	0.23	99.96	1.98	99.86	3.72	99.70	5.46
10........	100.00	0.29	99.96	2.04	99.86	3.78	99.69	5.52
12........	100.00	0.35	99.96	2.09	99.85	3.84	99.69	5.57
14........	100.00	0.41	99.95	2.15	99.85	3.90	99.68	5.63
16........	100.00	0.47	99.95	2.21	99.84	3.95	99.68	5.69
18........	100.00	0.52	99.95	2.27	99.84	4.01	99.67	5.75
20........	100.00	0.58	99.95	2.33	99.83	4.07	99.66	5.80
22........	100.00	0.64	99.94	2.38	99.83	4.13	99.66	5.86
24........	100.00	0.70	99.94	2.44	99.82	4.18	99.65	5.92
26........	99.99	0.76	99.94	2.50	99.82	4.24	99.64	5.98
28........	99.99	0.81	99.93	2.56	99.81	4.30	99.63	6.04
30........	99.99	0.87	99.93	2.62	99.81	4.36	99.63	6.09
32........	99.99	0.93	99.93	2.67	99.80	4.42	99.62	6.15
34........	99.99	0.99	99.93	2.73	99.80	4.48	99.62	6.21
36........	99.99	1.05	99.92	2.79	99.79	4.53	99.61	6.27
38........	99.99	1.11	99.92	2.85	99.79	4.59	99.60	6.33
40........	99.99	1.16	99.92	2.91	99.78	4.65	99.59	6.38
42........	99.99	1.22	99.91	2.97	99.78	4.71	99.59	6.44
44........	99.98	1.28	99.91	3.02	99.77	4.76	99.58	6.50
46........	99.98	1.34	99.90	3.08	99.77	4.82	99.57	6.56
48........	99.98	1.40	99.90	3.14	99.76	4.88	99.56	6.61
50........	99.98	1.45	99.90	3.20	99.76	4.94	99.56	6.67
52........	99.98	1.51	99.89	3.26	99.75	4.99	99.55	6.73
54........	99.98	1.57	99.89	3.31	99.74	5.05	99.54	6.78
56........	99.97	1.63	99.89	3.37	99.74	5.11	99.53	6.84
58........	99.97	1.69	99.88	3.43	99.73	5.17	99.52	6.90
60........	99.97	1.74	99.88	3.49	99.73	5.23	99.51	6.96
C = 0.75...	0.75	0.01	0.75	0.02	0.75	0.03	0.75	0.05
C = 1.00...	1.00	0.01	1.00	0.03	1.00	0.04	1.00	0.06
C = 1.25...	1.25	0.02	1.25	0.03	1.25	0.05	1.25	0.08

45.45.13

Figure 15-25.—Horizontal distances and elevations from stadia readings.

you use the same scale to lay off horizontal distances on the map.

Attach the paper to the table, and set up the table so that D_1 on the paper is directly over D_1 on the ground. With the edge of the alidade blade on D_1 and the telescope trained on A, orient the table so as to bring D_1A on the paper in line with the edge of the blade. Carefully level the table, and then check to ensure that the telescope is still trained on A, that the edge of the blade is still in line with D_1A on the paper, and that D_1 on the paper is still directly over D_1 on the ground.

Next carefully measure the vertical distance between the horizontal line of sight through the telescope and the ground level at D_1. Let's say this distance is 4.5 ft. This means that, whenever you sight on a rod, you will line up the horizontal crosshair with the 4.5-ft graduation on the rod.

Assuming that your alidade is equipped with a Beaman stadia arc (some alidades are otherwise equipped), the procedure for plotting point 1 of figure 15-24 is as follows. With the edge of the alidade blade exactly on D_1 on the paper, train the telescope on a rod held on point 1, and line the horizontal crosshair up with the 4.5-ft mark on the rod.

You read a rod intercept of 6.23 ft, which means a slope distance of 623.0 ft. On the H scale of the Beaman arc you read three-tenths of one percent; you'll have to estimate this less-than-one-percent reading. The horizontal distance, then, is three-tenths of one percent less than the slope distance, or 623.0 ft - (623.0 x 0.003 ft), or 623.0 - 1.87, which rounds off to the nearest foot at 621 ft. Add a focal distance of 1 ft, and you get the 622 ft shown in figure 15-24.

On the V scale you read 44. You know that the value you use is the difference between what you read and 50, which in this case is 6. Therefore, the difference in elevation is 6% of the slope distance, or 623.0 x 0.06, or 37.4 ft. Because the vertical angle was negative, the difference is subtracted. The elevation of point 1, then, equals the elevation of D_1 (532.4 ft) minus 37.4 ft, or 495.0 ft.

To plot the horizontal location of point 1, measure off 622 ft from D_1 (to the scale of the drawing, of course) along the blade of the alidade, mark the point and label it, and write in the elevation. Many topographers mark the point with the decimal point in the elevation.

BEAMAN STADIA ARC

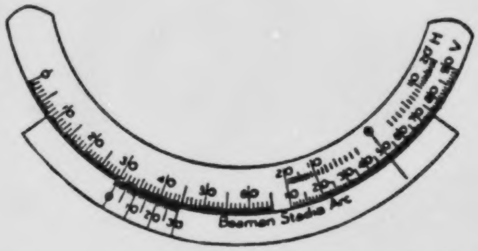
The Beaman stadia arc is a specially graduated arc on the vertical scale of the transit, figure 9-36, or on the planetable alidade, figure 15-26. The Beaman arc on the transit is also known as the STADIA CIRCLE. These arcs are used to determine distances and differences in elevation by stadia without using vertical angles and without using tables, diagrams or the stadia slide rule. A stadia arc has no vernier, but readings are indicated by index marks.

The stadia arc shown previously in figure 9-36, is the MULTIPLIER type (vertical index at zero); that is, the observed stadia distance is multiplied by the "Hor" stadia arc reading to get the horizontal distance, or the stadia distance is multiplied by the "Ver" stadia arc reading to obtain the vertical distance from the center of the instrument to the point sighted on the rod. This vertical distance, combined with the H.I. and the rod reading, will give the difference in elevation between the instrument station and the point where the rod is held.

The stadia arc shown in figure 15-26 is called the horizontal scale SUBTRACTION type (vertical index at 50.) The use of this type to obtain a horizontal distance and difference in elevation is explained in the following sections.

Horizontal Distance by Beaman Stadia Arc (Subtraction Type)

The H scale gives you a percentage which you can apply to an inclined stadia shot with the alidade to get the corresponding horizontal distance from the slope distance. Suppose that with the telescope inclined (that is, at a vertical angle other than 0°), you read an intercept of 2.45 ft on the stadia rod. The slope distance,



29.266M
Figure 15-26.—Vertical angle scale and Beaman stadia arc on an alidade.

then, is 2.45 x 100, or 245 ft. What is the corresponding horizontal distance? You read the graduation indicated by the Beaman arc indicator on the H scale, and find that the reading is 5. This means that the horizontal distance is 5% less than the slope distance, or 245 ft - (0.05 x 245 ft), or 245 - 12.25, or 232.7 ft.

Difference in Elevation by Beaman Stadia Arc (Vertical Index at 50)

The V scale on the Beaman arc is used to determine the difference in elevation between the elevation of the line of sight through the telescope (that is, the H.I.) and the elevation of the point you sighted on the level rod. Note that when the telescope is horizontal, the V scale on the Beaman arc reads 50. This arrangement makes the use of minus values unnecessary when you are sighting with the telescope at a negative vertical angle. To read the V scale, you take the difference between 50 and whatever you read on the scale, and apply this difference as follows to determine the difference in elevation.

Suppose that when you made the shot previously described (in which you read 5 on the H scale), the reading on the V scale was 71. In practice it is the custom to shoot the rod at a point which will give you an even reading on the V scale.

Because the reading was 71, the value you will use is 71 - 50, or 21%. This in turn means that the difference in elevation between the H.I. and the point you sighted on the rod is 21% of the slope distance. The slope distance in this case was 245.0 ft; therefore, the difference in elevation is 245.0 x 0.21, or 51.45 ft.

TOPOGRAPHIC SPECIFICATIONS

The scale and contour interval of the map for which you are surveying will be specified. These vary according to the purpose for which the map will be used. Obviously, a map which will be used for the rough design planning of a rural dirt road will be on a smaller scale, and have a larger contour interval, than one which will be used to guide constructors who are to erect a structure on a small tract in a built-up area.

The extent to which details must be shown may also be specified; if it isn't, it is usually inferred from the purpose of the map. The same applies to the accuracy and precision with which instrument points, details, and contour lines must be located.

The NAVFAC publication Drawings and Specifications, DM-6, contains general specifications for maps prepared under NAVFAC auspices. The following guidelines suggest the nature of typical map specifications.

A map should present legibly, clearly, and concisely a summation of all information pertinent to the use for which it is made, such as planning, design, construction, or record.

Topographic maps for preliminary site planning should preferably have a scale of 1 in. = 200 ft and a contour interval of 5 ft. Such maps should show all topographic features and structures, with particular attention given to boundary lines, highways, railroads, power lines, graveyards, large buildings or groups of buildings, shorelines, docking facilities, large rock strata, marshlands, and wooded areas. Secondary roads, small isolated buildings, small streams, and similar minor features are generally of less importance.

Topographic maps for detailed design for construction drawings should delineate all physical features, both natural and man-made, including underground structures. Scales commonly used are 1 in. = 20 ft, 1 in. = 40 ft, and 1 in. = 50 ft. Customary contour interval is 1 ft or 2 ft, depending on the character and extent of the project and the nature of the terrain. Besides contour lines, any spot elevations required to indicate surface relief should be shown.

Additional detail features which are usually required include:

1. Plane coordinates for grid systems, grid lines, and identification of the particular system or systems.
2. Directional orientation, usually indicated by North arrow.
3. Survey control, with ties to the grid system if there is one. (This means that the principal instrument stations from which details were located should be indicated in a suitable manner.)
4. All property, boundary, or right-of-way lines, with identification.

5. Roads and parking areas, including centerline location and elevation, curbs and gutters, and width and type of pavement.

6. Airport runways, taxiways, and apron pavements, including centerline locations with profile elevations and width and type of pavement.

7. Sidewalks and other walkways, with widths and elevations.

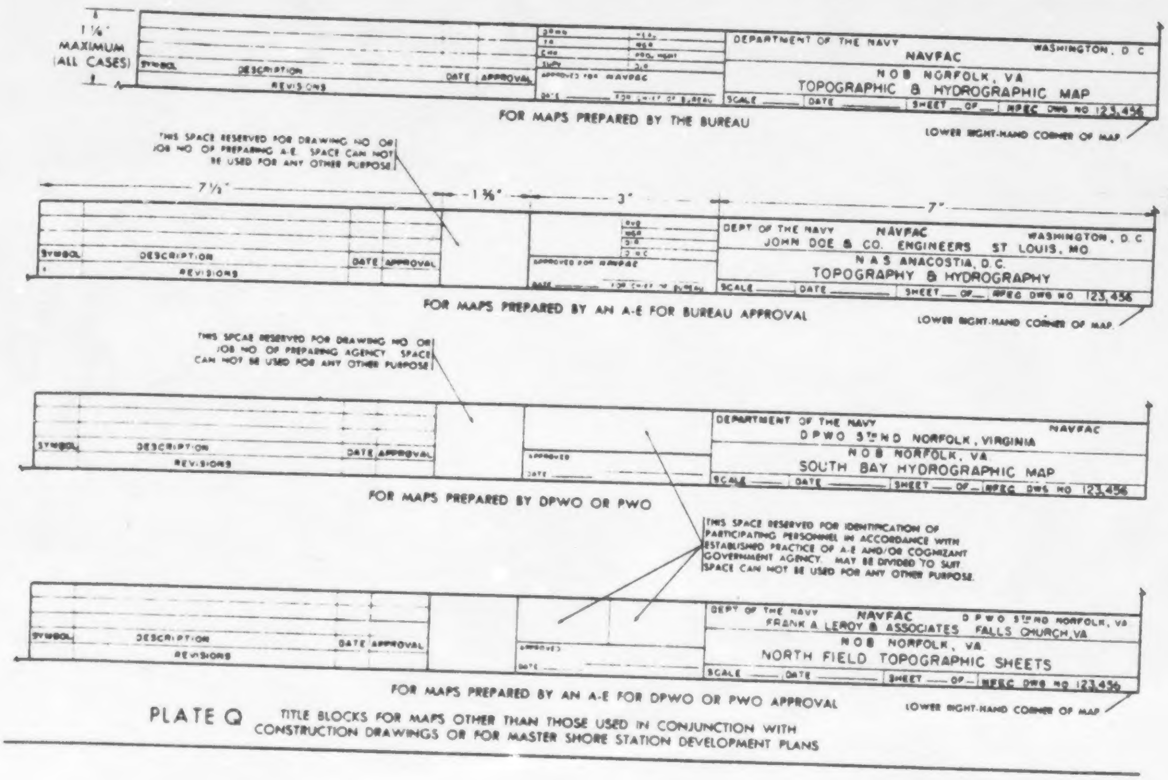
8. Railroads, including center line location, top-of-rail elevations, and any turnouts or crossovers.

9. Utilities and drainage facilities, such as power, gas, telephone, water, sanitary sewer, and storm sewer lines, including locations of all valve boxes, meter boxes, handholes, and manholes, and the invert elevations of sewers and appurtenances.

10. Locations, dimensions, and floor (usually first floor) elevations of all structures.

A uniform system of symbols is used to indicate various types of details. Some commonly used symbols are shown in figure 15-21. Every map must have a title block, and title blocks, too, must follow uniform standards. Four different types of sample title blocks for NAVFAC maps are shown in figure 15-27.

The precision with which horizontal and vertical control must be established may be specified. If not, it may be inferred from the purpose of the map. NAVFAC usually specifies that for a highway location map BMs be established at intervals of from 1000 to 1500 ft, as well as at bridge sites, intersections, and culvert sites. Level error of closure, in feet, must come to not more than $0.05\sqrt{M}$, where M is the length of the level run in miles. By NAVFAC definition, this amounts to third order precision. For


1 ⅛" MAXIMUM (ALL CASES)
SYMBOL
DESCRIPTION
DATE
APPROVAL
REVISIONS
APPROVED FOR NAVFAC
FOR CHIEF OF BUREAU
DEPARTMENT OF THE NAVY
NAVFAC
WASHINGTON, D. C.
N O B NORFOLK, VA
TOPOGRAPHIC & HYDROGRAPHIC MAP
SCALE
DATE
SHEET __ OF __
NFEC DWG NO 123,456
FOR MAPS PREPARED BY THE BUREAU
LOWER RIGHT-HAND CORNER OF MAP
THIS SPACE RESERVED FOR DRAWING NO OR JOB NO OF PREPARING A-E. SPACE CAN NOT BE USED FOR ANY OTHER PURPOSE
7 ½"
1 ⅜"
3"
7"
DEPT OF THE NAVY
JOHN DOE & CO. ENGINEERS ST LOUIS, MO
N A S ANACOSTIA, D.C.
TOPOGRAPHY & HYDROGRAPHY
FOR MAPS PREPARED BY AN A-E FOR BUREAU APPROVAL
THIS SPCAE RESERVED FOR DRAWING NO OR JOB NO OF PREPARING AGENCY. SPACE CAN NOT BE USED FOR ANY OTHER PURPOSE
DPWO 5TH ND NORFOLK, VIRGINIA
SOUTH BAY HYDROGRAPHIC MAP
APPROVED
FOR MAPS PREPARED BY DPWO OR PWO
THIS SPACE RESERVED FOR IDENTIFICATION OF PARTICIPATING PERSONNEL IN ACCORDANCE WITH ESTABLISHED PRACTICE OF A-E AND/OR COGNIZANT GOVERNMENT AGENCY. MAY BE DIVIDED TO SUIT SPACE CAN NOT BE USED FOR ANY OTHER PURPOSE
FRANK A LEROY & ASSOCIATES FALLS CHURCH, VA
D P W O 5TH ND NORFOLK, VA
NORTH FIELD TOPOGRAPHIC SHEETS
FOR MAPS PREPARED BY AN A-E FOR DPWO OR PWO APPROVAL
PLATE Q
TITLE BLOCKS FOR MAPS OTHER THAN THOSE USED IN CONJUNCTION WITH CONSTRUCTION DRAWINGS OR FOR MASTER SHORE STATION DEVELOPMENT PLANS

45.159
Figure 15-27.—Typical NAVFAC map title blocks.

horizontal control the equivalent would be a ratio of error of closure not to exceed 1/5000.

For detail location, both horizontal and vertical accuracy may be specified. A typical requirement for horizontal accuracy is that a planimetric feature on a map must be plotted within one-fiftieth of an inch of its true position on the map. A typical requirement for vertical accuracy is that the elevation of any contour point must be correct within one-fifth of the contour interval. The accuracy of the locations of contour lines may be checked by comparing two profiles, one scaled from the map, and the other run over the ground by differential leveling.

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branches on which you are likely to stand. Different kinds of wood vary greatly in strength. Oak, hickory, and elm trees, which have strong, flexible wood, are safer for climbing than (for example) poplar, catalpa, chestnut, or willow, which have soft or brittle wood. Limbs of all trees become brittle at low temperature—meaning that they break more easily in cold weather than they do in warm. Dead branches, or those containing many knots or fungus growths, are usually weak.

When standing on a limb, have your feet as close to the parent trunk as possible. Climb with special care when limbs are wet or icy. Wearing goggles when working in bushy trees may prevent eye injury. Before climbing a tree, ascertain whether any overhead wires pass through its foliage. If you must take a position in a tree within reach of live wire, place some sort of insulating safety equipment between yourself and the wire. Do not allow tree limbs to contact live wire, because moisture in a limb may cause a short circuit.

If you require cutting tools to clear a working space in a tree, haul them up with a handline, and lower them back down by the same device. Tools should never be thrown up into a tree or down onto the ground.

UNDERGROUND AND OVERHEAD LINES

Any below-street structure with an access opening (such as a manhole or a transformer vault) should be protected by a barrier or other suitable guard when the cover to the access opening is removed.

CROSSING ICE

Do not cross ice unless and until you are certain it will support your weight. Both the thickness and the nature of ice are important in determining its carrying capacity.

Because part of the supporting power of ice is derived from the water below it, a layer of ice which is in contact with the water surface is safer than one below which the water surface has fallen away.

An ice layer usually becomes thinner over current, near banks of streams or lakes, over warm springs, and over swampy ground. Rotten ice (which can be identified by its dull color and honeycomb texture) has little supporting power. Only light, clear ice is reliable.

NAVPERS 10634-B

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CHAPTER 16

ENGINEERING SURVEYS

An engineering survey forms the first of a chain of activities which will ultimately lead to a completed structure of some kind—such as a bridge, a building, or a highway. An engineering survey is usually subdivided into (1) a "design data" survey, and (2) a "construction" survey.

A typical design data survey is a "route" survey, which is itself subdivided into (1) the "reconnaissance" survey, (2) the "preliminary location" survey, and (3) the "final location" survey.

A construction survey is usually divided into (1) the "layout" or "stakeout" survey, and (2) the "as built" survey.

The techniques involved in the establishment of horizontal and vertical control, are explained in separate chapters. In this chapter a general description of a few common engineering-survey procedures are presented. There are a good many different varieties of construction projects, and no attempt can be made to refer to them all. In general, however, an engineering survey is most frequently concerned with the construction of a road/highway, a public utility (such as a sewer, power, or gas line) or a structure such as a building or a bridge.

HIGHWAY SURVEYS

How extensive the field and office work for a highway survey will be depends on the magnitude and complexity of the job. Some phases of the work may be done either in the field or in the office, and the choice here, and the exact procedures to be followed in all phases of the work, will be influenced by the number of men available and by the experience and capabilities of each.

FIELD WORK

The extent to which data is already available is the important factor in determining what field operations must be performed. The design and construction of a highway through an unmapped wilderness would probably require a reconnaissance survey, a preliminary location survey, a final location survey, and a construction survey. At the opposite extreme, a long-established Navy base (for example) might already have well-marked horizontal and vertical control networks and up-to-date topo maps available. In a case of this kind, neither a reconnaissance nor a preliminary survey might be required. The road could probably be designed on the basis of the already-existing design data, and the field work would begin with the final location survey.

Reconnaissance Survey

A reconnaissance survey provides data which enables design engineers to study the advantages and disadvantages of a variety of routes, and to determine which routes are feasible. You begin by procuring all existing maps which show the area to be reconnoitered. In reconnaissance, the study of existing maps is as important as the actual field work. The study of such maps (and of aerial photographs, if any) often eliminates an unfavorable route from further consideration, thus saving the reconnaissance field party much time and effort.

Contour maps give essential information on the relief of an area. Aerial photographs provide a quick means for preparing sketches and overlays valuable to a field party. Direct air observation gives an overview of an area which speeds up subsequent ground reconnaissance, whether the region is already mapped or not.

Begin the study of a map by marking the limits of the area to be reconnoitered and the specified terminals to be connected by the highway. Note whether or not there are any already-existing routes. Note ridge lines, water courses, mountain gaps, and similar control features. Look for terrain which will permit moderate grades without too much excavating, simplicity of alignment, and good balance of cuts and fills (meaning, a profile arrangement which makes it possible to fill depressions with cut taken from nearby high places).

Mark routes which seem to fit the situation, and which therefore should be reconnoitered in the field. From the map study, determine grades, estimate the amount of clearing required, and locate routes which will keep excavation to a minimum by taking advantage of terrain conditions. Mark stream crossings and marshy areas as possible locations for fords, bridges, or culverts.

The reconnaissance field party will follow the route or routes previously marked during the map study. Field reconnaissance presents the opportunity for checking the actual conditions on the ground, and for noting any discrepancies in the maps or aerial photographs. Make notes of soil conditions, availability of construction materials (such as sand or gravel), unusual grade or alignment problems, and requirements as to clearing and grubbing. Take photographs or make sketches of reference points, control points, structure sites, terrain obstacles, landslides, washouts, or any other unusual circumstances.

A reconnaissance survey party usually carries lightweight, not-overly-precise instruments. Directions and angles may be determined by compass. Approximate elevations may be determined by aneroid barometer or altimeter. An Abney hand level (clinometer) may be used to estimate elevations and to project level lines. Other useful items are pocket tapes, binoculars, pedometer and pace tallies, cameras, watches, maps, and field notebooks.

It is important to keep design considerations in mind while running a reconnaissance survey. Remember that future operations may require the further expansion of the route system presently being designed. Locate portions of the new route, whenever possible, along already-existing roads or trails. Locate on stable, easily drained, high-bearing-strength soils. Avoid swamps, marshes, low-bearing-strength soils, sharp curves, and routes requiring large amounts of earth-moving.

Keep the need for bridges and drainage structures to a minimum. When the tactical situation permits, locate roads in forward combat zones where they will be concealed and protected from enemy fire.

The report turned in by the reconnaissance field party must be as complete as possible, because it provides the major data which makes the selection of the most feasible route or routes possible.

Preliminary Survey

A preliminary survey is a more detailed study of one or more routes tentatively selected on the basis of a reconnaissance survey report. It consists essentially of the surveying and mapping of a strip of land along the centerline of a tentatively selected route.

In a preliminary survey you run a traverse (sometimes called a "P-line" or "survey base line"), establish bench marks, record topography, run profiles, and take cross-sections. For many projects, the preliminary survey may be conducted by a transit party alone. For others there may be three or more parties, as, for example: a transit party, a level party, and a topographic party.

The preliminary survey may be plotted while the party is in the field. This practice provides a more accurate representation of the terrain, reduces the possibility of error, and enables you to resolve doubtful situations while you have the actual terrain under observation. However, on the basis of complete data (notes, sketches, and the like), office personnel can prepare plots of the surveyed lines and grades.

Final Location and Construction Layout Survey

The final location (usually called just the "location") and construction layout surveys are named together because they constitute a continuous operation. The location survey consists of establishing the approved layout in the field. It is an instrument survey and it provides the alignment, grades, and locations which will guide the constructors. It consists of setting stakes to mark the limits of earthmoving operations, to locate structures, and to establish final grades and alignments. Part of this is location, part of it is construction layout, and the continuous operation goes on from the setting of stakes prior to any construction work right through to the end of the actual construction.

Prior to the final location survey, office studies consisting of the preparation of a map from preliminary survey data, projection of a tentative alignment and profile, and preliminary estimates of quantities and costs are made and used as guidance for the final location phase.

The final location in the field is carefully established by the transit party using the paper location prepared from the preliminary survey. The centerline may vary from the paper location due to objects or conditions that were not previously considered, but these changes should not be made by the surveyor without the authority of the engineering officer. Stations are marked; levels are run; and grades established. All construction lines are based on the final centerline.

The surveyors must be ahead of the construction activity, both time-wise and distance-wise, to avoid delays in the work. Details of stakeout procedures are explained later in this chapter.

PRINCIPLES OF HORIZONTAL CURVES

As you know, the basic layout of a highway is composed of a series of connected straight lines (an open traverse). A change in horizontal direction is the point of intersection (P.I.) of two straight lines. If you were to just follow these intersecting straight lines in the actual construction of the highway, the changes in direction would be too abrupt and the road would be unsafe for modern, high speed vehicles. It is, therefore, necessary to interpose circular curves between them to smooth out the highway. With the introduction of these curves, these straight lines are called "TANGENTS."

Practically all modern highway curves are a combination of transition spiral curves and circular curves. The radius determines the sharpness or the flatness of a spiral curve. In modern, high speed highways, we prefer very flat curves, which have greater radius. To make vehicle travel even more safe and comfortable on highways, transition spiral curves or easement curves are introduced just before and after the circular curves.

In highwork work, the curves needed for the location or improvement of small secondary roads may be worked out in the field. Usually, however, the horizontal curves are computed after the route has been selected, the field surveys have been done, and the survey base line and necessary topographic features have been plotted. In urban work, the curves of streets are designed as an integral part of the preliminary and final layouts which are usually done in the drawing room. In highway work, the road itself is the end result and purpose of the design; but in urban work the streets and their curves are of secondary importance, and the best utilization of the building sites is of primary importance.

The design of the curve consists principally of selecting the length of the radius or "degree of curvature," (explained later). This selection is based on such considerations as the design speed of the highway and the sight distance as limited by headlights or obstructions (see fig. 16-1). Typical radii which you may encounter are 40,000 feet on an interstate highway, 1,000 feet on a major thoroughfare in a city, 500 feet on an industrial access road, and 150 feet on a minor residential street.

Type of Horizontal Curves

There are four types of horizontal curves. They are:

1. Simple. The simple curve is an arc of a circle (view A, fig. 16-2). The radius of the circle determines the sharpness or flatness of the curve.
2. Compound. Frequently, the terrain will necessitate the use of the compound curve. This curve normally consists of two simple curves joined together, both curving in the same direction, (view B, fig. 15-2).
3. Reverse. A reverse curve consists of two simple curves joined together, but curving in opposite directions. For safety reasons, this

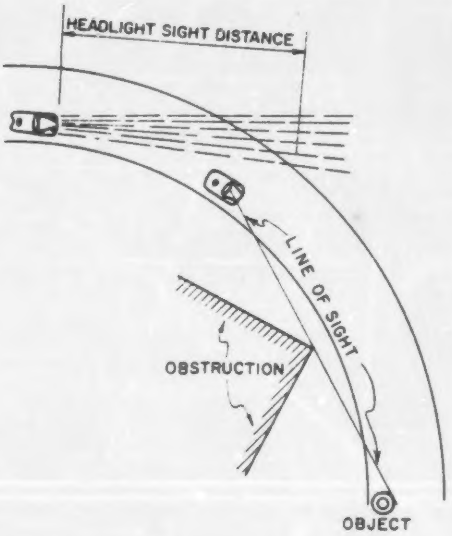

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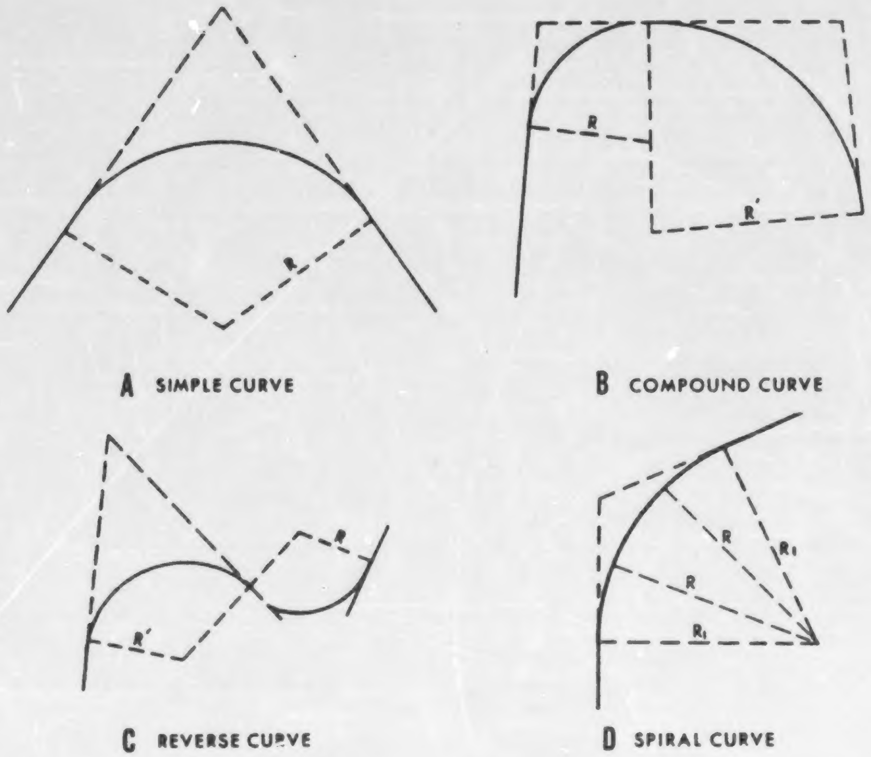
Figure 16-1. — Lines of sight.



A SIMPLE CURVE
B COMPOUND CURVE
C REVERSE CURVE
D SPIRAL CURVE

45.797
Figure 16-2. —Horizontal curves.

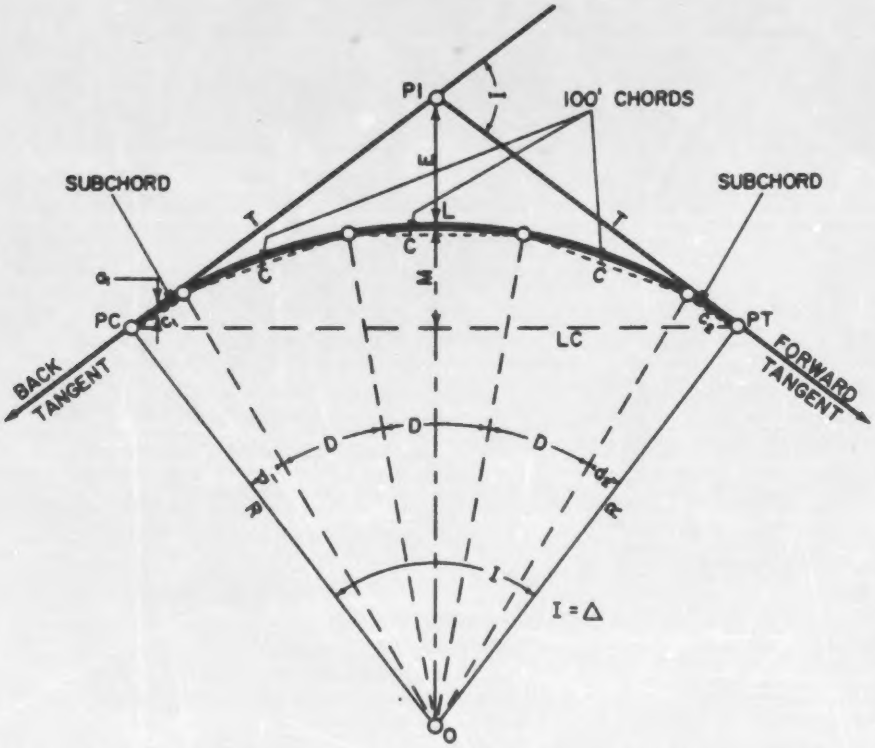
curve should not be used unless absolutely necessary (view C, fig. 16-2).

4. Spiral. The spiral is a curve which has a varying radius. It is used on railroads and most modern highways. Its purpose is to provide a transition from the tangent to a simple curve or between simple curves in a compound curve (view D, fig. 16-2).

Elements of a Simple Curve

During the preliminary survey for a highway, it is customary for the surveyor to number the stations from the beginning of the project, forward. For example: 0 + 00 indicates the beginning of the project; 15 + 52.96 would indicate a point, 1,552.96 feet from the beginning. A full station is 100 feet. 15 + 00 and 16 + 00 are full stations. A plus station indicates a point between full stations. 15 + 52.96 is a plus station. Horizontal curve is introduced whenever the route changes direction so that it is important that you must know the elements of this curve and how to compute for an element when other elements are given. The elements of a simple curve are shown in figure 16-3; an explanation of each element follows.

PI	POINT OF INTERSECTION. The point of intersection is the point where the back and forward tangents intersect. It is indicated by the initials PI. It is one of the stations on the preliminary traverse.
I	The INTERSECTING ANGLE. The intersecting angle is the deflection angle at the PI. Its value is either computed from the preliminary traverse angles or measured in the field. It is indicated by the initial I.
R	The RADIUS. The radius is the radius of the circle of which the curve is an arc.



82.158
Figure 16-3.—Elements of a simple curve.

PC	The POINT OF CURVATURE. The point of curvature, indicated by the initials PC, is the point where the circular curve begins. The back tangent is tangent to the curve at this point.
POC	POINT ON CURVE. This is any point along the curve and is indicated by the initials POC.
PT	THE POINT OF TENGENCY. The point of tangency is the end of the curve. It is indicated by the initials PT. The forward tangent is the tangent to curve at this point.
L	THE LENGTH OF CURVE. The length of curve is the distance from the PC to the PT measured along the curve.
T	THE TANGENT DISTANCE. The tangent distance is the distances along the tangents from the PI to the PC or PT. These distances are equal on a simple curve.
Δ	The CENTRAL ANGLE. The central angle is the angle formed by two radii drawn from the center of the circle (0) to the PC and PT. The central angle is equal in value to the I angle. Some authorities call both the intersecting angle and central angle either I or Δ.
LC	LONG CHORD. The long chord is the chord from the PC to the PT.
E	EXTERNAL DISTANCE. The external distance is the distance from the PI to the midpoint of the curve. The external distance bisects the interior angle at the PI.
M	MIDDLE ORDINATE. The middle ordinate is the distance from the midpoint of the curve to the midpoint of the long chord.

The extension of the middle ordinate bisects the central angle.

D DEGREE OF CURVE. The degree of curve defines the "sharpness" or "flatness" of the curve. There are two definitions commonly used for degree of curve. (See fig. 16-4.) They are:

1. Arc Definition. The arc definition states that the degree of curve is the angle formed by two radii drawn from the center of the circle (Point 0, fig. 16-3) to the ends of an arc 100 feet long. In this definition the degree of curve and radius are inversely proportional. For example: D:360°:: Arc:Circumference. Substituting $D = 1°$, we obtain $1°:360°::100:2\pi R$ or $1:360°::100:6.283185308R$, therefore $R = 36{,}000 \div 6.283185308 = 5729.58$ feet. If the degree of curve is 5 degrees, the radius is 1145.92 feet. As the degree of curve increases the radius decreases. It should be noted that for a given intersecting angle or central angle, all of the parts of the curve are inversely proportional to the degree of curve, when using the arc definition. This definition is used primarily for highways.

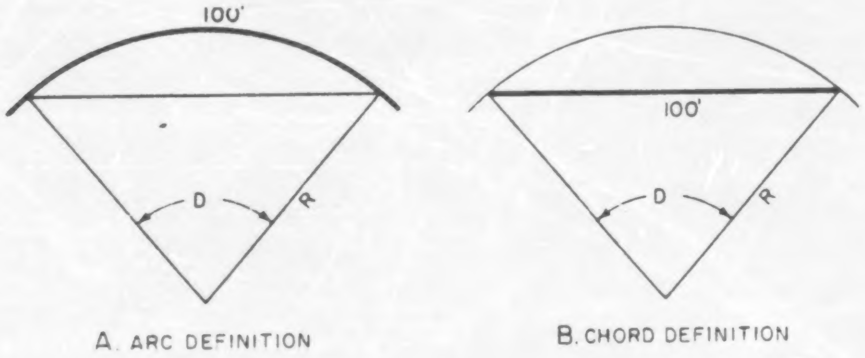
2. Chord definition. The chord definition states that the degree of curve is the angle formed by two radii drawn from the center of the circle to the ends of a chord 100 feet long. The radius is computed by the formula $50/\sin 1/2\ D = R$. Assuming D to be 1 degree and substituting in the formula we get: $50/\sin 0° 30' = 50/0.0087265355 = 5729.65$ feet. If D is 5 degrees, the radius is 1146.28 feet. Notice that the larger the degree of curve, the "sharper" the curve and the shorter the radius. However, the radius and degree of curve ARE NOT inversely proportional when using the chord definition. The chord definition is used primarily on railroads in civilian practice, and is used by the military for both roads and railroads.

CHORDS. On curves with long radii, it is impractical to stake the curve by locating the center of the circle and swinging the arc with a tape. These curves are laid out by staking the ends of a series of chords (fig. 16-5). Since the ends of the chords lie on the circumference of the curve, the arc is then defined in the field. The length of the chords will vary with the degree of curve. To reduce the discrepancy between the arc distance and chord distance, the following chord lengths are commonly used:

0° to 3° degree of curve — 100 feet
over 3° to 8° degree of curve — 50 feet
over 8° to 16° degree of curve — 25 feet
over 16° degree of curve — 10 feet

The chord lengths above are the maximum distances in which the discrepancy between the arc length and chord length will fall within the allowable error for taping, which is .02' per 100 feet on most construction surveys. Depending upon the terrain and the needs of the project foreman, the curve may be staked out with shorter or longer chords than the recommended.

DEFLECTION ANGLES. The deflection angles are the angles between a tangent and the ends of chords, from the PC. They are used to locate the direction in which the chords are to be laid out. The total of the deflection angles is always equal to on half the I angle. This total serves as a check on the computed deflection angles.



A. ARC DEFINITION

B. CHORD DEFINITION

45.799

Figure 16-4. — Degree of curve.

Simple Curve Formulas

The following formulas are used in the computation of a simple curve. All of the formulas apply to both the arc and chord definitions except those noted.

$R = \frac{5729.58}{D}$ (Arc Definition) (1)

$R = \frac{50}{\sin 1/2\ D}$ (Chord Definition) (2)

$T = R \tan 1/2\ I$ (3)

$L = 100 \frac{I}{D}$ (Exact for the arc deflection.) (4)

The distance around the chords for the chord definition.

$PC = PI - T$ (5)

$PT = PC + L$ (6)

$E = R \text{ Exsec } 1/2\ I$ (7)

$E = T \tan 1/4\ I$ (8)

$E = \frac{R}{\cos 1/2\ I} - R$ (9)

$M = R - (R \cos 1/2\ I)$ (10)

$M = R \text{ Vers } 1/2\ I$ (11)

Deflection angles. 1. $\left(\frac{D}{2}\right)\left(\frac{C}{100}\right)$ — Exact for arc definition. Approximate for the chord definition. This formula gives an answer in degrees.
2. 0.3 CD — Exact for the arc definition. Approximate for the chord definition. This formula gives an answer in minutes. (This is No. 1 above multiplied by 60 to get result in minutes.)
3. Sine of deflection angle = $\frac{C}{2R}$. Exact for chord definition.
(12)

$LC = 2 R \sin 1/2\ I$ (13)

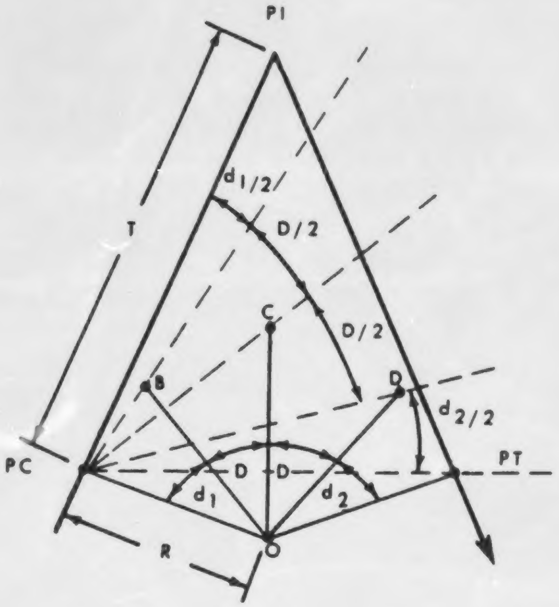


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Figure 16-5. — Deflection angles.

Solution of a Simple Curve

To solve a simple curve, three elements must be known. Two of these elements must be the PI and I angle. Normally the third part will be the degree of curve which is given in the project specifications or computed using one of the elements which has been limited by the terrain.

The PI and I angle are normally determined on the preliminary traverse for the road, but may also be determined by triangulation when the PI is inaccessible.

In any situation three parts must be known, the point of intersection, the intersecting angle, and the degree of curve, before the complete curve can be computed. Assume that the following is known: PI = 18 + 00, I = 75°, and D = 15°. The curve is solved by both the arc and chord definitions as follows:

Arc Definition

$R = \frac{5729.58}{D} = \frac{5729.58}{15°} = 381.97$

$T = R \text{ Tan } 1/2 \text{ I}$

$T = 381.97 \times 0.76733 = 293.10$

$PI - T = PC$

$18 + 00 = PI$

$\underline{293.10 \quad = T}$

$15 + 06.90 = PC$

$L = 100 \text{ I/D}$

$L = 100 \times \frac{75}{15} = 500'$

$PC + L = PT$

$15 + 06.90 = PC$

$\underline{500.00 = L}$

$20 + 06.90 = PT$

$E = R \text{ Exsec } 1/2 \text{ I}$

$E = 381.97 \times 0.26047 = 99.49$

$M = R \text{ Vers } 1/2 \text{ I}$

$M = 381.97 \times 0.20665 = 78.93$

$LC = 2 R \sin 1/2 \text{ I}$

$LC = 2 \times 381.97 \times 0.60876 = 465.06$

Chord Definition

$R = \frac{50}{\text{Sin } 1/2 \text{ D}} = \frac{50}{0.13053} = 383.05$

$T = R \text{ Tan } 1/2 \text{ I}$

$T = 383.05 \times 0.76733 = 293.93$

$PI - T = PC$

$18 + 00 = PI$

$\underline{293.93 \quad = T}$

$15 + 06.07 = PC$

$L = 100 \text{ I/D}$

$L = 100 \times \frac{75}{15} = 500'$

$PC + L = PT$

$15 + 06.07 = PC$

$\underline{500.00 = L}$

$20 + 06.07 = PT$

$E = R \text{ Exsec } 1/2 \text{ I}$

$E = 383.05 \times 0.26047 = 99.77$

$M = R \text{ Vers } 1/2 \text{ I}$

$M = 383.05 \times 0.20665 = 79.16$

$LC = 2 R \text{ Sin } 1/2 \text{ I}$

$LC = 2 \times 383.05 \times 0.60876 = 466.37$

Since the degree of curve is 15 degrees, the chord length would normally be 25 feet as stated earlier. However, the following computations will be based on a staking interval of 50'. It is customary for the first stake after the PC to be placed at a plus station divisible by the chord length. The centerline of the road is normally staked at intervals of 50 or 100 feet between curves. By placing the first stake after the PC at a plus station divisible by chord length, the stationing does not confuse the level party when profile levels are run on the centerline. The first stake after the PC for this curve will be at station 15 + 50, therefore the first chord length or subchord will be 43.10 feet for the arc definition and 43.93 feet for the chord definition. Similarly there will be a subchord at the end of the curve from station 20 + 00 to the PT. This chord will be 6.90 feet and 6.07 feet for the arc and chord definitions respectively. The subchord at the beginning is designated C1 and the end, C2 (fig. 16-3).

After the subchords have been determined, the deflection angles are computed, using the formulas given above. Technically, the formulas for the arc definitions are not exact for the chord definition; however, when a one-minute transit is used in staking the curve, they may be used for either definition. The deflection angles are:

Arc Definition

$d_1 = 0.3 \text{ CD}$

$d_1 = 0.3 \times 43.10 \times 15° = 3° \, 13.95'$

$d_2 = 0.3 \times 6.90 \times 15° = 0° \, 31.05'$

Chord Definition

$d_1 = 0.3 \text{ CD}$

$d_1 = 0.3 \times 43.93 \times 15° = 3° \, 17.685'$

$d_2 = 0.3 \times 6.07 \times 15° = 0° \, 27.315'$

The deflection angle for each chord of 50 feet is computed and found to be 0.3 x 50 x 15 = 3° 45'. Since there are 9 chords of 50 feet, the sum of the deflection angles for 50-foot chords is 9 x 3° 45' = 33° 45'. A convenient method of calculating the full chords is to remember that the deflection angle equals 1/2D for 100-foot chords; 1/4D for 50 feet; and 1/8D for 25 feet.

The sum of d_1, d_2, and the deflections for the full chords are:

Arc Definition	Chord Definition
$d_1 = 3° \, 13.95'$	$d_1 = 3° \, 17.685'$
$d_2 = 0° \, 31.05'$	$d_2 = 0° \, 27.315'$
$d_{50} = 33° \, 45'$	$d_{50} = 33° \, 45.00'$
Total 37° 30.00'	Total 37° 30.00'

Note that the total of the deflection angles is equal to one-half of the I angle. If the tota deflection does not equal one-half of I, a mistake has been made in the calculations. After the total deflection has been decided, the angles are determined for each station on the curve. In this step they are rounded off to the least reading on the instrument to be used in the field. For this problem assume that a one-minute instrument is to be used. The deflection angles are then:

Arc Definition

Station	Chord	Deflection angle
PC 15+06.90	------	------------
15+50	C_1 43.10	d_1 3°14'
16+00	50	6°59'
16+50	50	10°44'
17+00	50	14°29'
17+50	50	18°14'
18+00	50	21°59'
18+50	50	25°44'
19+00	50	29°29'
19+50	50	33°14'
20+00	50	36°59'
PT 20+06.90	C_2 06.90	37°30'

Chord Definition

Station	Chord	Deflection angle
PC 15+06.07	------	---------------
15+50	C_1 43.93	d_1 3°18'
16+00	50	7°03'
16+50	50	10°48'
17+00	50	14°33'
17+50	50	18°18'
18+00	50	22°03'
18+50	50	25°48'
19+00	50	29°33'
19+50	50	33°18'
20+00	50	37°03'
PT 20+06.07	C_2 6.07	37°30'

Note that the deflection angle for each station is an accumulative total.

A comparison of the two definitions will show a difference in the tangent distance, radii, and deflection angles. Although the lengths are the same, the length on the arc definition is the actual arc distance while the length on the chord definition is around the chords.

The curve that was just solved had an I angle and degree of curve whose values were whole degrees. When the I angle and degree of curve consist of degrees and minutes, the procedure in solving the curve does not change, but care must be taken in substituting these values into the formulas for length and deflection angles. For example: I = 42° 15', D = 5° 37'. The minutes in each angle must be changed to a decimal part of a degree. To obtain the required accuracy they should be converted to five decimal places. An alternate method for computing the length is to convert the I angle and degree of curve to minutes, thus, 42°15' = 2,535 minutes and 5°37' = 337 minutes. Substituting into the length formula gives:

$$L = 100 \times \frac{2535}{337} = 752.23 \text{ feet.}$$

This method gives an exact result. By converting the minutes to a decimal part of a degree to nearest five places, the same result is obtained.

Since the total of the deflection angles should be one-half of the I angle, a problem arises when the I angle contains an odd number of minutes and the instrument used is a one-minute transit. Since the PT is normally staked prior to running the curve, the total deflection will be a check on the PT. Therefore, it should be computed to the nearest 0.5 degree. If the total deflection checks to the nearest minute in the field it can be considered to be correct.

The computation of simple curves is simplified by the use of tables. The use of tables for this purpose is explained thoroughly in EA 1 & C.

Simple Curve Layout

To layout the simple curve (arc definition) just computed above, the usual procedure is as follows:

1. With the instrument at the PI, the instrumentman sights on the preceding PI or at a distant station and keeps the chainman on line while the tangent distance is measured to locate the PC. After the PC has been staked, the instrument is then trained on the forward PI and the PT is located.

2. The instrument is then set up at the PC and the angle from the PI to the PT is measured. This angle should be equal to one-half the I

angle; if not, either the PC or PT has been located in the wrong position.

3. With the first deflection angle (3° 14') set on the plates, the instrumentman keeps the chainman on line as the first subchord distance (43.10') is measured from the PC.

4. Without touching the lower motion, the second deflection angle (6° 59') is set on the plates. The chainmen measure the chord from the previous station while the instrumentman keeps the head chainman on line.

5. The succeeding stations are staked out in the same manner. If the work is done correctly, the last deflection angle will point on the PT and the distance will be the subchord length (6.90') from the last station prior to the PT.

When it is impossible to stake out the entire curve from the PC, an adaptation of the above procedure is used. Stake out the curve as far as possible from the PC. If for some reason, a station cannot be seen from the PC, move the transit forward and set up over a station along the curve. Pick a station for a backsight and set the deflection angle for that station on the plates. Sight on this station with the telescope in the reverse position. Plunge the telescope and set the remainder of the stations as if the transit was set over the PC. If the setup in the middle of the curve has been made and it is still impossible to set the next stake due to some obstruction, the remainder of the curve can be "backed in." To back in a curve, occupy the PT. Sight on the PI and set 1/2 the I angle on the plates. The transit is now oriented so that if the PC is observed the plates will read zero, which is the deflection angle shown in the notes for that station. The curve stakes can then be set in the same order shown in the notes or in the reverse order. Remember to use the deflection angles and chords from the top of the column or from the bottom of the column. Although it has been set up as a method to miss obstructions, the backing in method is very widely used as a method of laying out curves. The method is to proceed to the approximate midpoint of the curve by laying out the deflection angles and chords from the PC and then laying out the remainder of the curve from the PT. By using this method, any error in the curve is in the center where it is less noticeable.

The methods of laying out curves with various obstruction problems are explained in EA 1 & C.

OFFICE WORK

After the type and general location of a highway have been decided and after the necessary design data have been obtained in the field, there are a number of office tasks to be performed. These tasks include:

a. Plotting the plan view.
b. Plotting the profile.
c. Plotting the alignment.
d. Designing the gradients.
e. Plotting the cross-sections.
f. Determining end-areas.
g. Computing volumes of cut and fill.

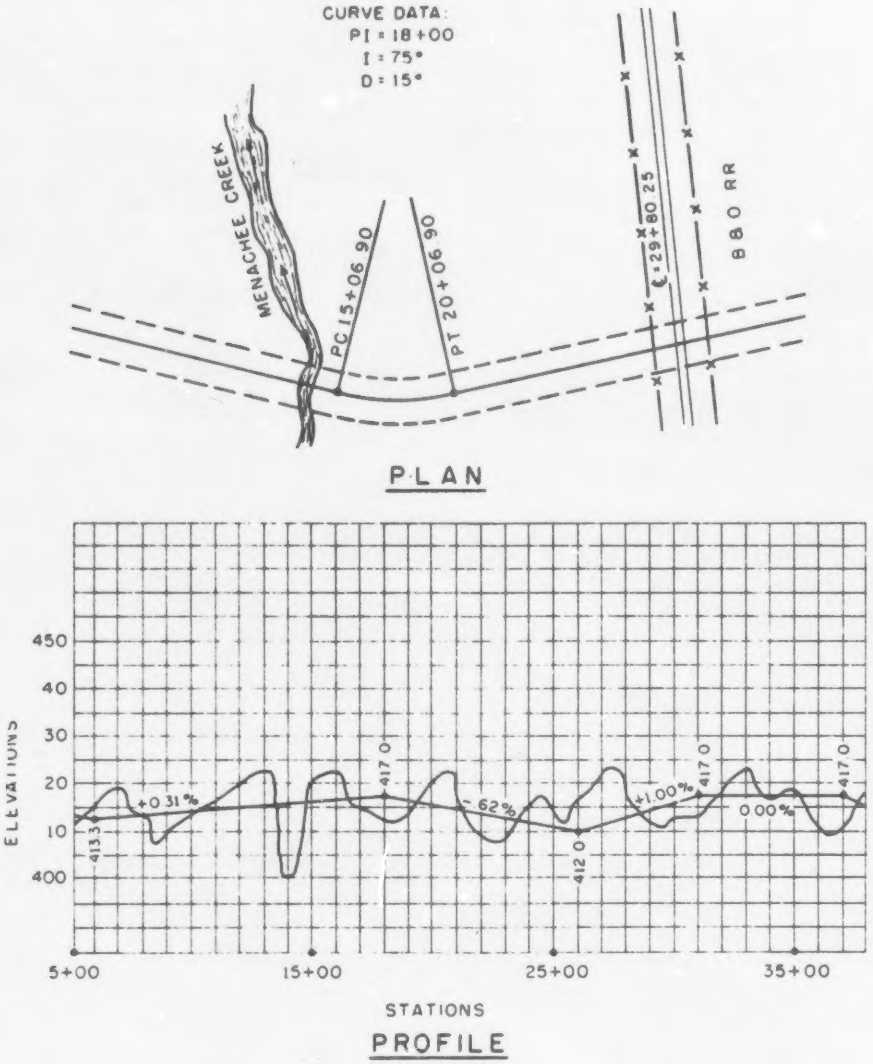
These operations may be repeated one or more times as trial designs are developed and then revised or discarded. For highway plan and profile we can plot on the same sheet. State Highway Department and Bureau of Public Roads (B.P.R.) used Standard Form called "Federal Aid Plan Profile Sheet." Figure 16-6 shows a plotted highway plan and profile view. Plotting cross-sections is discussed later in this chapter.

Plotting the Plan View

Plotting the plan view of a highway is similar to plotting a traverse, except for the introduction of the curves and curve data. The important elements of the curve, such as the PI, I, D and the like, which are necessary for curve computation are shown in the form of notes at each curve point. (See the plan view, fig. 16-6.) As you can see in the drawing, topographic details are also included.

Profile Plotting

Profile plotting is usually done on regular "profile paper," consisting of paper ruled with horizontal and vertical parallel lines as shown in figure 16-6. Vertical lines are spaced 1/4 in. or 1/2 in. apart; horizontal lines are spaced 1/20 or 1/10 in. apart. In figure 16-6 the vertical lines on the original paper (reduced in size for reproduction in this book) were 1/4 in. apart. On the original paper there was a horizontal line at every 1/20 in. interval; for the sake of



45.801
Figure 16-6. — Plan and profile for a highway.

clarity, only those at every 1/4-in. interval have been reproduced.

The first consideration in profile plotting is to select suitable horizontal and vertical scales for the profile paper. The suitability of scales varies with the character of the ground and other circumstances. In figure 16-6 the horizontal scale used was 1 in. = 400 ft and the vertical scale used was 1 in. = 20 ft (reduced in size for reproduction in this book). Normally, to facilitate plotting, the chosen scales must be proportional numbers in multiples of ten such as those given above (H, 1" = 400' and V, 1" = 20'). The stations and elevations may be written as shown in figure 16-6.

The profile is usually plotted from profile level notes, though it may be plotted from elevations obtained from contour lines. Assume that profile level notes indicate the following centerline elevations at the following stations from 5 + 00 through 15 + 00.

Station	Elevation (ft)
5 + 00	411.9
6 + 00	415.0
7 + 00	417.8
8 + 00	412.0
8 + 75	406.9
9 + 00	411.0
10 + 00	413.2
10 + 50	413.5
11 + 00	415.9
12 + 00	417.3
13 + 00	423.0
13 + 80	412.0
14 + 00	402.0
15 + 00	418.2

As you can see, an elevation was taken at every full station, and also at every plus where there was a significant change in elevation. You can see how important it is to follow this last procedure. If an elevation had not been taken at 8 + 75, the drop which exists between 8 + 00 and 9 + 00 would not show on the profile.

If you check through the listed elevations, you will see how each of them was plotted as a point located where a vertical line indicating the station intersects a horizontal line indicating the elevation of that station. Note, too, that it is the custom to label the stations where the line crosses highways, streams, and railroads.

Besides the profile of the existing terrain, the vertical tangents of the proposed highway centerline have been plotted in. The end-elevation for each of these (that is, the elevations of the P.V.I.'s, or points of vertical intersection) were determined by the design engineers. Various circumstances were considered, one of the important ones being to facilitate as much as possible the filling of each depression with an approximately equal volume of cut taken from a nearby hump, or from two nearby humps.

The gradient, in terms of percentage of slope (total rise or fall in feet per 100 horizontal feet), is marked on each of the vertical tangents. This percentage is computed for a tangent as follows. For the tangent running from station 6 + 00 to station 18 + 00 the total rise is the difference in elevation, or 417.0 - 413.3, or +3.7 ft. The horizontal distance between the stations is 1200 ft. The percentage of slope, then, is the value of x in the equation

$$\frac{1200.0}{3.7} = \frac{100}{x}, \text{ or } 0.31\%.$$

For a tangent running from station 18 + 00 to station 26 + 00, the total slope downward is the difference in elevation, or 412.0 - 417.0, or -5.0 ft. The distance between the stations is 800 ft. The percentage of slope then, is the value of x in the equation

$$\frac{800}{-5.0} = \frac{100}{x}, \text{ or } -0.62\%.$$

Types of Cross-Sections

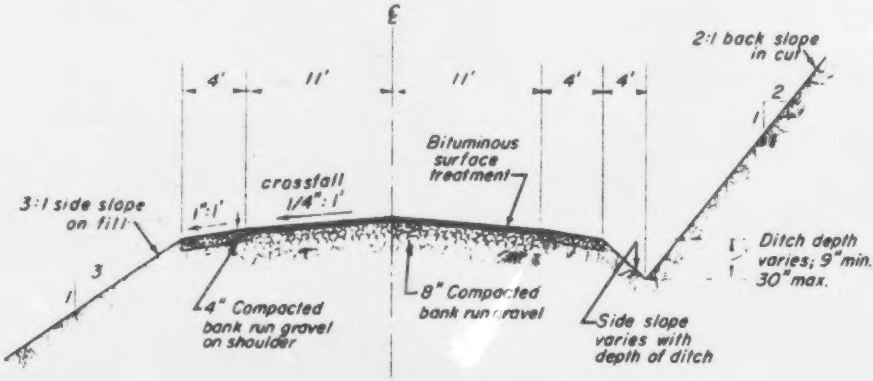
Figure 16-7 shows a typical design cross-section. As you can see, just about everything you need to know to construct the highway (including the materials to be used and their thicknesses) is given here.

However, this design section is a section of the completed highway. For the purpose of stakeout and for earth-moving calculations, the cross-section line of the existing ground at each successive station must be plotted and the design data cross-section (typical section of the highway) is then superimposed.

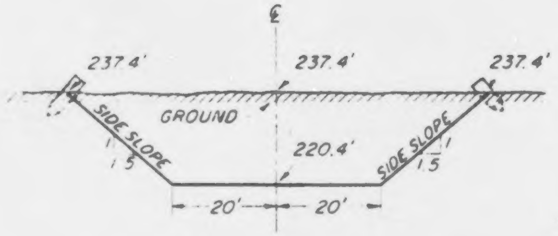
Figure 16-8 shows a simplified cross-section of a 40-ft wide highway. The elevation of the existing surface is 237.4 ft all the way across; therefore this is what is called a "level" section. Finished grade for the highway at this station (that is, the proposed centerline elevation for the finished highway surface) is 220.4 ft. The prescribed "side-slope ratio" is 1.5:1 — that is, a horizontal unit of 1.5 for every 1 unit of vertical rise.

Because the ground line across the cross-section is level and the side-slope ratio the same on both sides, the horizontal distance from the centerline to the point where the side-slope will meet the natural surface will be the same on both sides. A "slope stake" is driven at this point to guide the earth-movers. The horizontal distance from the centerline to a slope stake can be computed by processes to be explained later.

In the case of this cross-section you know (1) the width of the highway, (2) the side-slope ratio, and (3) the proposed finished grade. Besides this, all you need to know to set slope stakes is the ground elevation of the slope-stake point on each side. As this elevation is the same


℄
4'
11'
11'
4'
4'
2:1 back slope in cut
2
1
Bituminous surface treatment
crossfall 1/4":1'
3:1 side slope on fill
1":1'
3
1
Ditch depth varies; 9" min., 30" max.
4" Compacted bank run gravel on shoulder
8" Compacted bank run gravel
Side slope varies with depth of ditch

45.375
Figure 16-7.—Typical design cross-section.



45.376
Figure 16-8.—Level section in cut.

on both sides in this case, only a single level shot for elevation is needed. For this reason, a section of this kind is called a one-level (or, for short, just a level) section. Because the entire sectional area consists of material to be excavated or CUT, it is called a SECTION IN CUT.

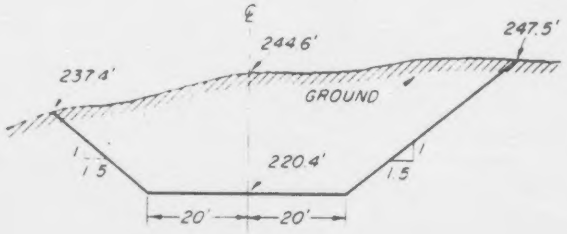
In the section shown in figure 16-9 the ground line across the section is sloping rather than level. Therefore, to plot this section you would need three different elevations: one for the left slope stake, one for the centerline grade stake, and one for the right slope stake. If these three levels are taken, such a section is called a three-level section in cut. If additional levels are taken midway between the centerline and the slope stake on either side, it is called a five-level section in cut. In either case it is a section "in cut" because the entire cross-section area consists of cut.

Level, three-level, and five-level sections are called regular sections.

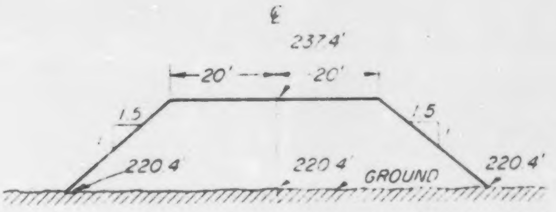
Figure 16-10 shows a level section in fill; figure 16-11 shows a three-level section in fill. The section shown in figure 16-12, consisting partly of cut and partly of fill, is called a side-hill section.

When a more accurate picture of cross-sections than can be obtained from regular sections is desired, irregular sections are taken and plotted. For an irregular section you take, besides the regular levels, additional levels on either side of the centerline. You take these at set intervals and at major breaks in the ground line.

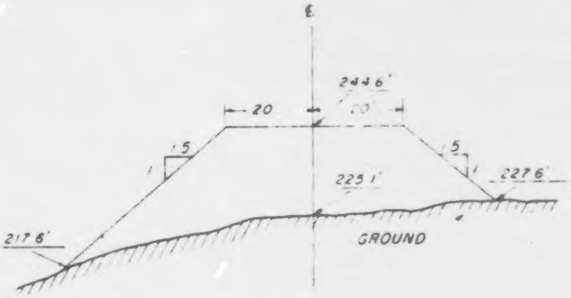
Cross-sections may be preliminary or final. Preliminary cross-sections (from the P-line or survey base line) are irregular sections which are plotted before the finished grade has been determined. They may be obtained by levels run



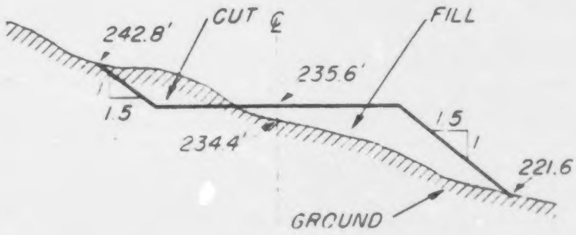
45.377
Figure 16-9.—Three level section in cut.



45.380
Figure 16-10. — Level section in fill.



45.381
Figure 16-11. — Three level section in fill.



45.382
Figure 16-12. — Side-hill section.

in the field, or by elevations picked off contour lines on a topo map.

Final cross-sections are sections of the finally designed road. They may be prepared in the same manner as preliminary sections, or they may be regular sections plotted from field data obtained after the finished grade has been set. The term "final" cross-section is also applied to as-built or "pay" sections taken after construction is completed.

Plotting Cross-Sections

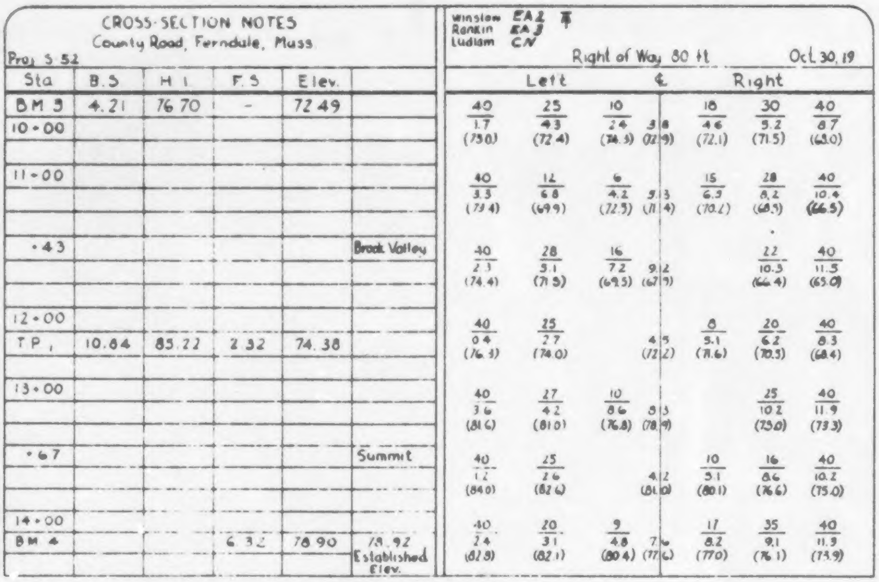
Cross-sections are usually plotted on cross-section paper, which comes either in rolls or sheets. It is ruled into 1-in. squares with heavy orange or green lines, and with lighter lines into 1/10 in. squares. Cross-section paper is commonly called 10" x 10" paper.

Plot each cross-section separately and below each plot show the station number. Place the first cross-section at the top of a sheet and continue downward until all the sections have been plotted. Two or more sections may be plotted on the same sheet. In a major highway project, cross-sections are plotted in a continuous roll of cross-section paper. Some surveyor's prefer to plot the cross-sections from bottom to top of the paper. They may also prefer to record cross-section notes in the same manner. If you follow these methods of plotting and recording, you are properly oriented with the actual direction of the highway, that is, your left is also towards the left of the highway; it is also the left of the cross-section notes and the plotted cross-section. Really, it does not matter which way to follow as long as you are properly oriented at all times.

Unlike profile plotting, in cross-section plotting it is often the case that the same scale is used for both vertical and horizontal distance. Common scales are 1" = 5' and 1" = 10'. When sections are shallow, however, it is best to exaggerate the vertical scale, making it from two to ten times the horizontal scale.

For the centerline for a row of sections use one of the heavier vertical lines on the paper, far enough away from the margin to ensure that no plot will run outside the limits of the paper. Note the depths indicated for the first section to be plotted, and select a horizontal line for the base which is far enough below the top margin. Mark this with the base elevation. Then lay off the horizontal distances of the section surface elevations on either side of the centerline, and plot the elevations according to the level data. Finally, connect these plotted points by using a straightedge or freehand-drawn lines.

Figure 16-13(A) shows cross-section notes for the existing ground along a proposed road. In figure 16-13(B) the sections at station 11 + 00 and 11 + 43 have been plotted. The field party took, for each station, the ground elevation 40 ft to right and left of the centerline. For each station, however, the centerline distance of the intermediate elevations varies. Therefore, these are irregular sections.

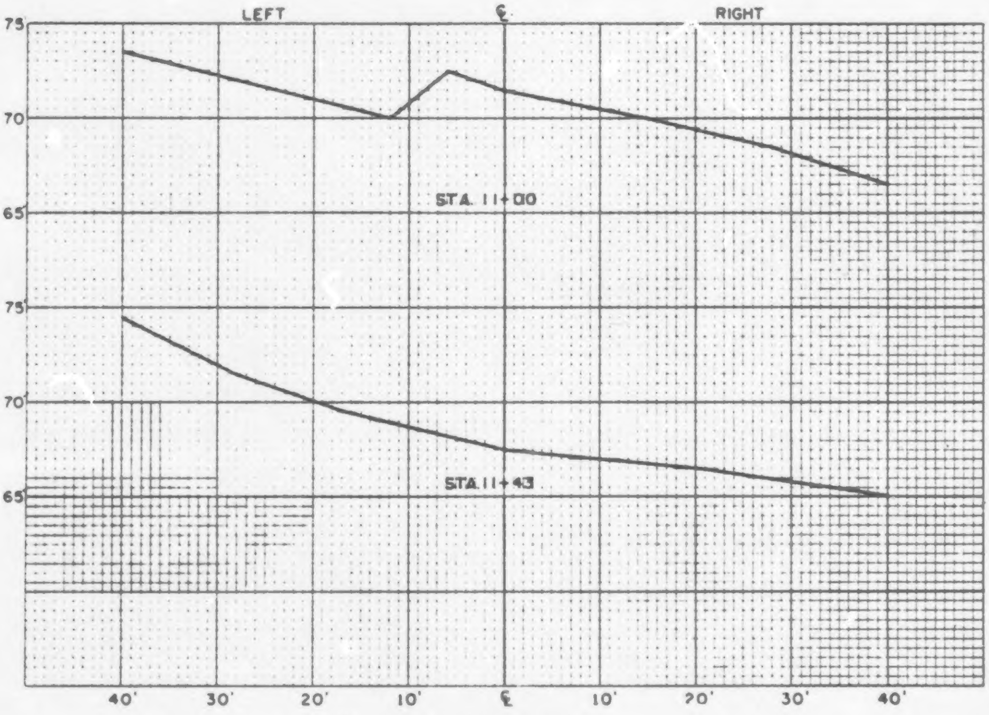


CROSS-SECTION NOTES
County Road, Ferndale, Mass.
Proj. S 52

Winslow EA2 ⊼
Rankin EA3
Ludlam CN
Right of Way 80 ft Oct. 30, 19

Sta	B.S	H.I	F.S	Elev.		Left			℄	Right		
BM 3	4.21	76 70	-	72 49								
10+00						40 1.7 (73.0)	25 4.3 (72.4)	10 2.4 (74.3)	3.8 (72.9)	18 4.6 (72.1)	30 5.2 (71.5)	40 8.7 (68.0)
11+00						40 3.3 (73.4)	12 6.8 (69.9)	6 4.2 (72.5)	5.3 (71.4)	15 6.5 (70.2)	28 8.2 (68.5)	40 10.4 (66.5)
+43					Brook Valley	40 2.3 (74.4)	28 5.1 (71.5)	16 7.2 (69.5)	9.2 (67.9)		22 10.3 (66.4)	40 11.5 (65.0)
12+00						40 0.4 (76.3)	25 2.7 (74.0)		4.5 (72.2)	8 5.1 (71.6)	20 6.2 (70.5)	40 8.3 (68.4)
TP1	10.84	85.22	2.32	74.38								
13+00						40 3.6 (81.6)	27 4.2 (81.0)	10 8.6 (76.8)	8.3 (78.9)		25 10.2 (75.0)	40 11.9 (73.3)
+67					Summit	40 1.2 (84.0)	25 2.6 (82.6)		4.2 (81.0)	10 5.1 (80.1)	16 8.6 (76.6)	40 10.2 (75.0)
14+00						40 2.4 (82.8)	20 3.1 (82.1)	9 4.8 (80.4)	7.6 (77.6)	17 8.2 (77.0)	35 9.1 (76.1)	40 11.3 (73.9)
BM 4			6.32	78.90	78.92 Established Elev.							

A CROSS-SECTION NOTES



B CROSS-SECTIONS PLOTTED

45.383

Figure 16-13.—A. Cross-section notes. B. Cross-sections plotted.

For both of the stations plotted the H.I. was 76.70 ft. For the point 6 ft left of centerline at station 11 + 00 you see the 4.2 written below the 6. This was the reading obtained from a rod held on this point. The number 72.5 shown in bracket right below the number 4.2 is an elevation of this point. The elevation is obtained by subtracting from the H.I. the rod reading F.S. (76.70 - 4.20, or 72.50). You can see this point plotted in 6 ft to the left of the centerline and at an elevation of 72.5 ft in figure 16-13(B). Now, if the notes are reduced in the office, the general practice is to print the elevations in RED, that is, the elevation just computed (72.5), will appear in red in the cross-section notes (view A of fig. 16-13).

After the road gradients (either preliminary or final) have been designed, the design data cross-section may be plotted on the existing ground line section plot at each station to complete the picture of the end-area as it will exist in the finished highway. You obtain the finished grade elevation for each station from the profile. Plot the finished grade point (usually on the centerline) at each cross-section. Then draw in the outline of the pavement surface, ditches, and cut or fill slopes as they show on the typical design section. Plotting may be done with triangles, but a faster method is to use templates made of plastic, thin wood, sturdy cardboard, or other suitable material. Prepare templates for a cut section, a fill section, and a side-hill section (which may be flipped over to accommodate the direction of hillside slope).

The procedures just described are the most common, and pertain to irregular sections. However, if regular sections have been taken in the field after the gradients have been designed, then both the existing and the finished surfaces will be plotted. Field notes for simplified 3-level sections on a highway are shown in figure 16-14. On the data side the profile elevation and the grade elevation at each station are listed. In the columns headed "left" and "right" on the remarks side, the upper numbers with the appropriate letter symbols ("C" for Cut, "F" for Fill) are the CUTS or FILLS, and the lower numbers are the distances out from the center. These values indicate points at which the slope stakes are driven. If a five-level or irregular section is being recorded, the other points must be written between those for the center and the slope stakes.

These field notes give you coordinates from which you can plot sections as shown in figure 16-15. In that figure, only the lines at every 1/4-in. interval are shown, for purposes of clarity. The scale, both horizontal and vertical, is 1 in. = 10 ft.; therefore, the interval between each pair of lines represents 2.5 ft.

The highway is to be 40 ft wide; therefore, the edge of the pavement will for each plotted section lie 8 squares (8 x 2.5 = 20) on either side of the centerline. Figure 16-14 shows that, for station 305, the left-hand slope stake is located 29.8 ft from the centerline and 8.2 ft above grade. The right-hand slope stake is located 35.3 ft from the centerline and 12.3 ft above grade. You can see how the locations of these stakes can be plotted in after you have selected an appropriate horizontal line for the grade line, and how the side slopes can then be drawn in.

The ground line at the centerline is 9.3 ft above grade. You plot a point here, and then finish the plot of the section by drawing lines from the centerline point to the two slope stake points.

You would plot a 5 level section in exactly the same way, except for the fact that additional ground points between the centerline and the slope stakes would be plotted in.

AREAS OF CROSS-SECTIONS

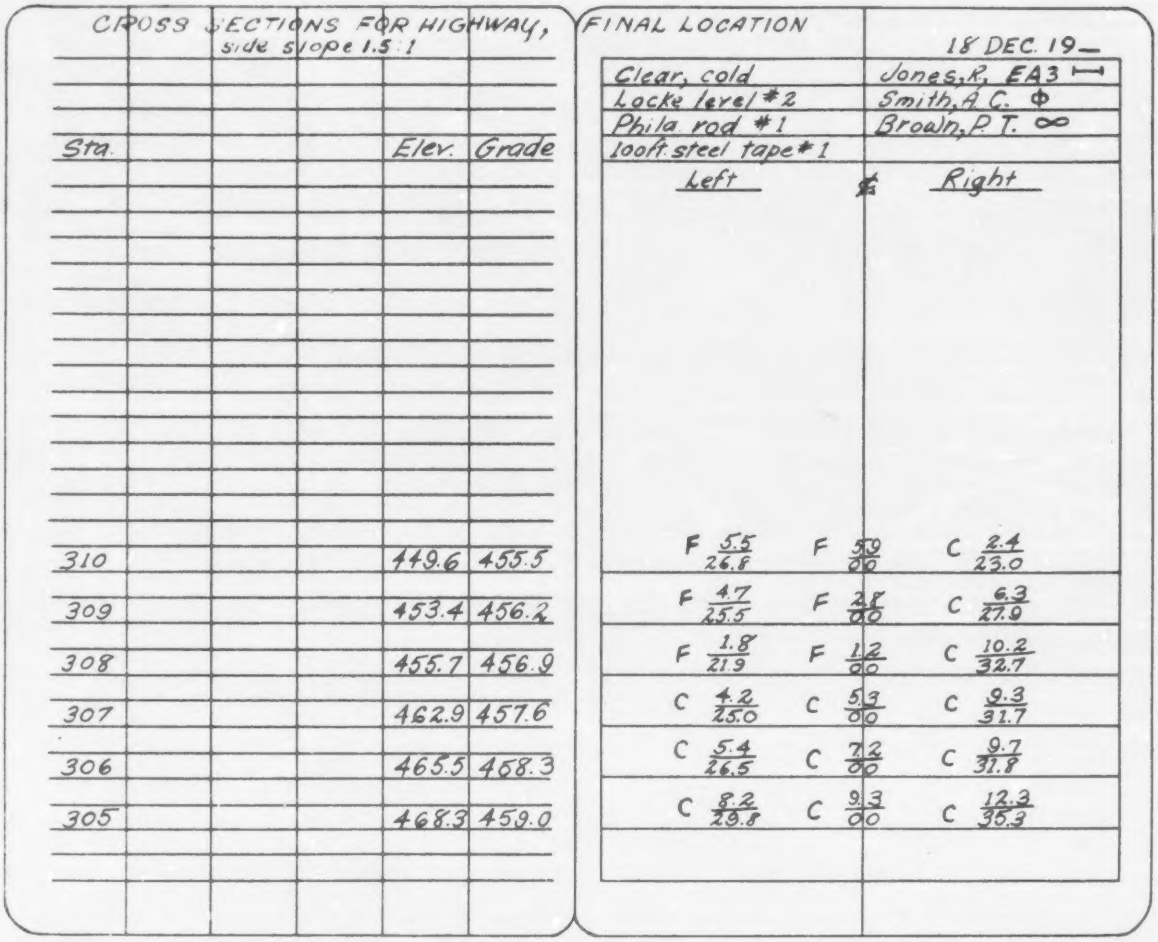
Suppose that for a highway excavation you want to know the volume of cut which must be taken out between two stations. If you know the areas of the cross-sections at each station and the horizontal distance between the stations, the volume of cut amounts to the product of the average between the end areas ("average end area") times the distance between the stations.

The easiest way to determine the area of a cross-section is to run a planimeter around the plotted outline of the section. Another method is by "counting the squares" in a plotted section, as explained in chapter 5, "Level and Traverse Computations," EA 1 & C training manual.

Resolving into Triangles

The area of any polygon, regular or irregular, can be determined by resolving the polygon into triangles (any polygon can be thus resolved), solving each triangle for area, and then determining the sum of the areas.

Take, for example, the plot of station 305 + 00 shown in figure 16-16. Figure 16-17 shows how this figure can be resolved into two triangles,



CROSS SECTIONS FOR HIGHWAY, side slope 1.5:1

Sta.				Elev.	Grade
310				449.6	455.5
309				453.4	456.2
308				455.7	456.9
307				462.9	457.6
306				465.5	458.3
305				468.3	459.0

FINAL LOCATION

18 DEC. 19—

Clear, cold	Jones, R. EA3 ⟼
Locke level #2	Smith, A.C. Φ
Phila rod #1	Brown, P.T. ∞
100ft. steel tape #1	

Left	℄	Right
F $\frac{5.5}{26.8}$	F $\frac{5.9}{0.0}$	C $\frac{2.4}{23.0}$
F $\frac{4.7}{25.5}$	F $\frac{2.8}{0.0}$	C $\frac{6.3}{27.9}$
F $\frac{1.8}{21.9}$	F $\frac{1.2}{0.0}$	C $\frac{10.2}{32.7}$
C $\frac{4.2}{25.0}$	C $\frac{5.3}{0.0}$	C $\frac{9.3}{31.7}$
C $\frac{5.4}{26.5}$	C $\frac{7.2}{0.0}$	C $\frac{9.7}{31.8}$
C $\frac{8.2}{29.8}$	C $\frac{9.3}{0.0}$	C $\frac{12.3}{35.3}$

45.7.12

Figure 16-14.—Field notes for three-level cross-sections.

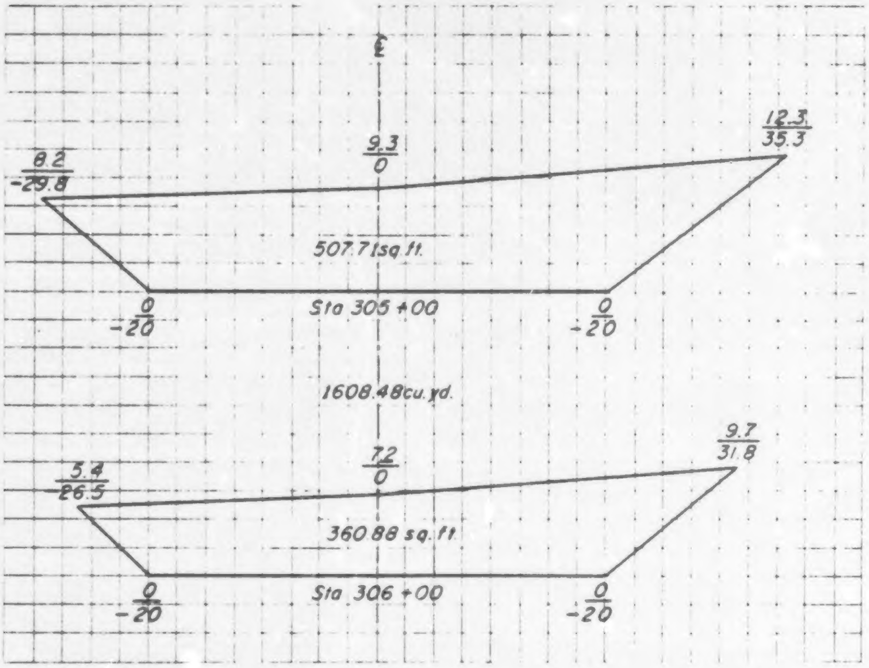
ABH and DFE, and two trapezoids, BCGH and CGFD. For each of these figures the dimensions have been approximately determined by the scale of the plot. The area of each trapezoid equals the length of the base times the average altitude. Using this formula, the area of BCGH comes to 183.0 sq ft, and the area of CGFD comes to 203.0 sq ft. For the area of each of the triangles you can use the formula, $A = \sqrt{s(s-a)(s-b)(s-c)}$. The area of triangle ABH comes to 42.5 sq ft and the area of triangle DFE comes to 84.4 sq ft. Therefore, the total area of a section of sta. 305 is 180 + 203 + 42.5 + 84.4, or 509.9 sq ft.

Area by Formula

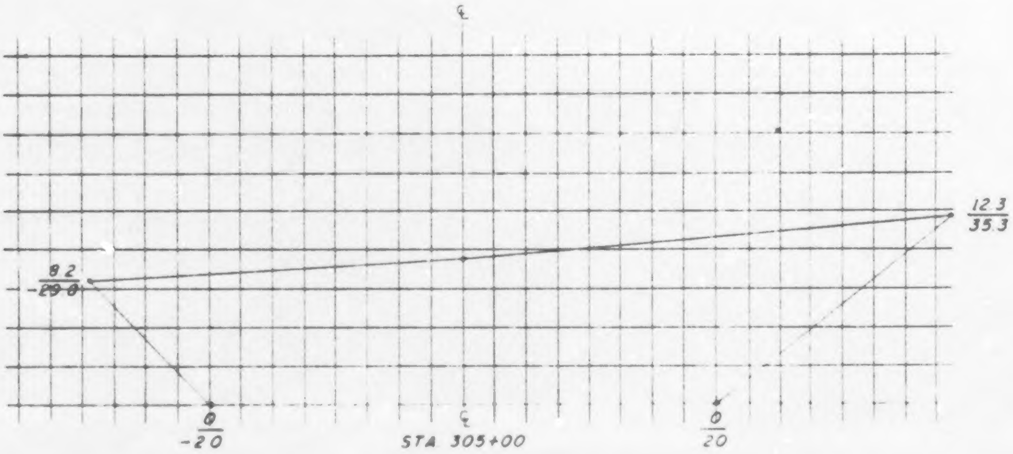
A regular section area for a 3-level section can be more exactly determined by applying the following formula:

$$A = \frac{W}{4}(h_l + h_r) + \frac{c}{2}(d_l + d_r)$$

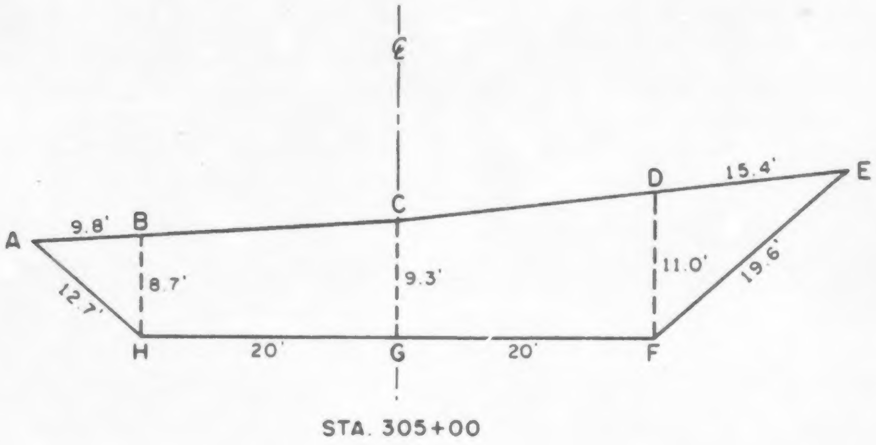
W in this formula is the width of the highway; h_l and h_r are the vertical distance of the left and right slope stakes above grade; d_l and d_r are the centerline distances of the left and right slope stakes; and c is the depth of



45.385
Figure 16-15.—Cross-section plots of stations 305 and 306 of highway in figure 16-14.



45.684
Figure 16-16.—A cross-section plotted on cross-section paper.



45.685
Figure 16-17.—Station 305 + 00 cross-sectional area resolved into triangles and trapezoids.

the centerline cut or fill. Applying the formula for station 305 + 00, you get:

$$A = \frac{40}{4}(8.2 + 12.3) + \frac{9.3}{2}(29.8 + 35.3) = 507.71 \text{ sq ft.}$$

Using the same formula for station 306 + 00 you get 360.88 sq ft. Note that the volume of cut lying between the stations (which are presumed to be 100.0 horizontal feet apart) is inscribed between the plots in figure 16-15. By **average-end-area** this works out as follows:

$$V = \frac{507.71 + 360.88}{2} \times 100 = 434.29 \times 100 = 43429 \text{ cu ft.}$$

Earth volume is usually given in cubic yards. There are 27 cu ft in a cu yd; therefore, the volume of cut between 305 + 00 and 306 + 00 in cu yds equals

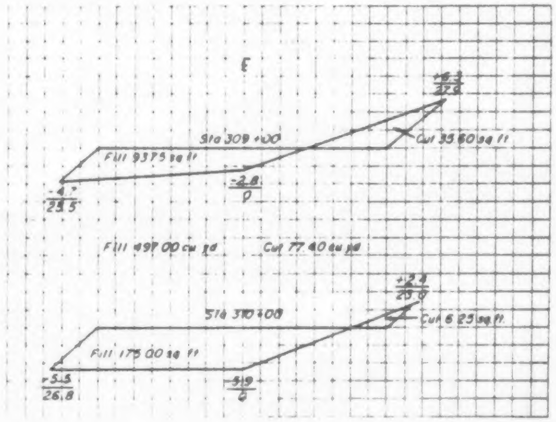
$$\frac{43429}{27}, \text{ or } 1608.48 \text{ cu yd.}$$

The sections at stations 305 + 00 and 306 + 00 are sections in cut. From the notes shown in figure 16-14, however, you can see that the sections at 308 + 00 and 309 + 00 must be partly in cut and partly in fill, or "side hill" sections.

The method of plotting is the same, as shown in figure 16-18. To get the section areas here you can "count the squares." Each square represents 2.5 x 2.5, or 6.25 sq ft. There are about 15 squares in the fill area of the section at 309 + 00, which comes to 15 x 6.25, or 93.75 sq ft. There are about 5.7 squares in the cut area at this station, which makes 5.7 x 6.25, or 35.62 sq ft.

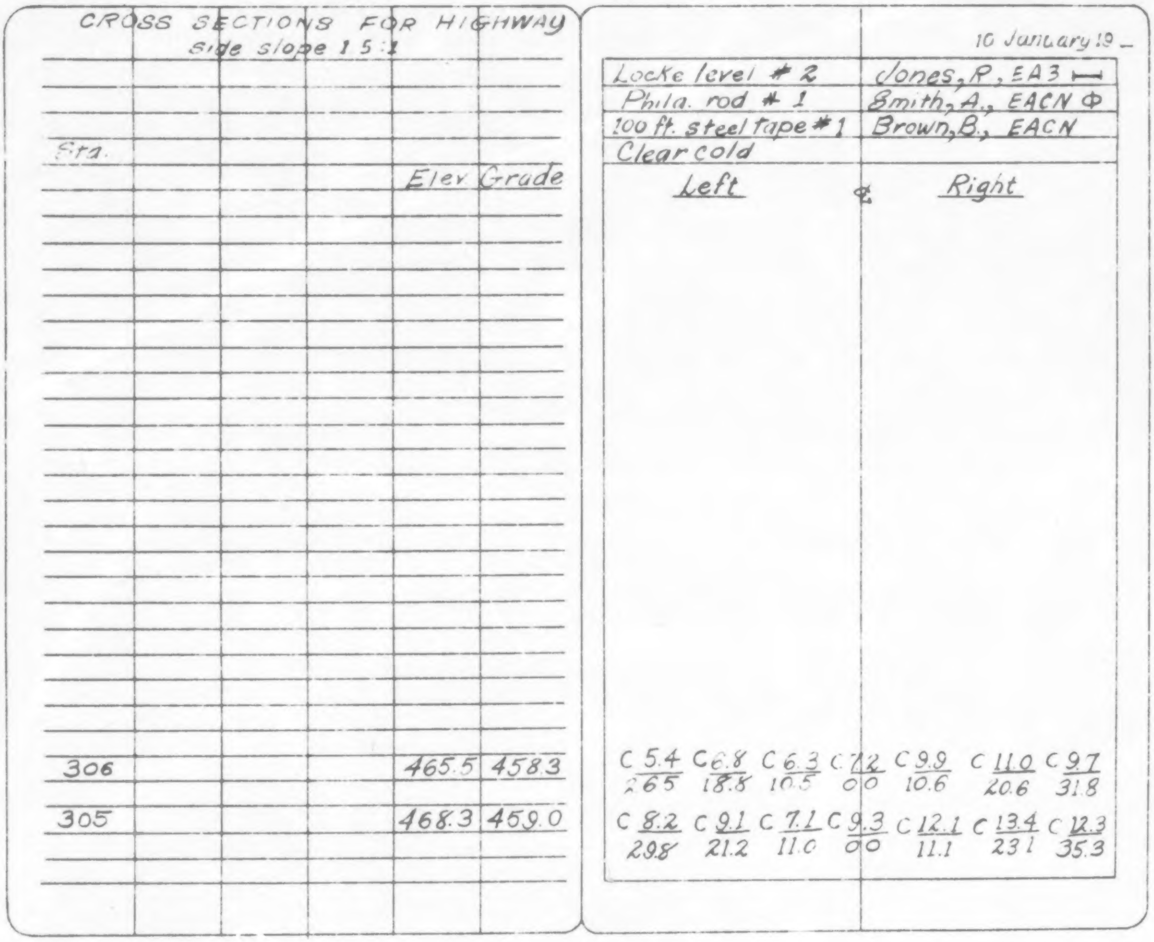
Area of 5-Level or Irregular Section

Figure 16-19 shows notes for two irregular sections, and figure 16-20 shows the two sections plotted.



45.386
Figure 16-18.—Plots of side-hill sections 309 and 310 noted in figure 16-14.

359



CROSS SECTIONS FOR HIGHWAY
Side slope 1.5:1

10 January 19 –

Locke level # 2	Jones, R., EA3 ⟝
Phila. rod # 1	Smith, A., EACN Φ
100 ft. steel tape # 1	Brown, B., EACN
Clear cold	

Sta.	Elev	Grade	Left			℄	Right		
306	465.5	458.3	C $\frac{5.4}{26.5}$	C $\frac{6.8}{18.8}$	C $\frac{6.3}{10.5}$	C $\frac{7.2}{0.0}$	C $\frac{9.9}{10.6}$	C $\frac{11.0}{20.6}$	C $\frac{9.7}{31.8}$
305	468.3	459.0	C $\frac{8.2}{29.8}$	C $\frac{9.1}{21.2}$	C $\frac{7.1}{11.0}$	C $\frac{9.3}{0.0}$	C $\frac{12.1}{11.1}$	C $\frac{13.4}{23.1}$	C $\frac{12.3}{35.3}$

45.7.13

Figure 16-19.— Field notes for irregular sections.

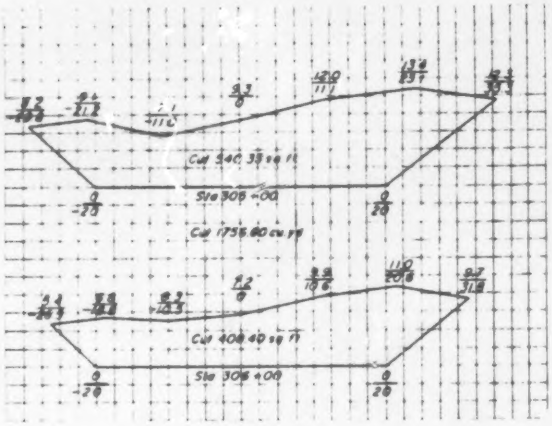
To determine the area of a section of this kind you can use a method of determining area by coordinates. First, you consider the point where the centerline intersects the grade line as the point of origin for the coordinates. Vertical distances above or below the grade line are positive Y coordinates; vertical distances below the grade line are negative Y coordinates. A point on the grade line itself has a Y coordinate of 0.

Similarly, horizontal distances to the right of the centerline are positive X coordinates; horizontal distances to the left of the centerline are negative X coordinates; and any point on the centerline itself has an X coordinate of 0.

First, plot the cross section as shown in figure 16-20. Be sure that the X and Y coordinates have their proper signs. Then, starting at a particular point and going successively in a clockwise direction, write down the coordinates as follows:

$$\frac{0}{-20.0} \nearrow \frac{8.2}{-29.8} \times \frac{9.2}{-21.2} \times \frac{7.1}{-11.0} \times \frac{9.3}{0.0} \times \frac{12.0}{11.1} \times \frac{13.4}{23.1} \times \frac{12.3}{35.3} \times \frac{0}{20.0} \nwarrow \frac{0}{-20}$$

Then multiply each UPPER term by the ALGEBRAIC difference of the FOLLOWING lower



45.388
Figure 16-20.—Cross-section plots of stations 305 and 306 noted in figure 16-19.

term and the PRECEDING lower term as indicated by the direction of the arrows above. The algebraic sum of these products is the double area of the cross section. Proceed with the computation as follows:

8.2 [-21.2 - (-20.0)]
9.1 [-11.0 - (-29.8)]
7.1 [0 - (-21.2)]
9.3 [11.1 - (-11.0)]
12.0 [23.1 - 0]
13.4 [35.3 - 11.1]
12.3 [20.0 - 23.1]

This works out to the following solution for double area:

8.2 x (-1.2) = -9.8
9.1 x (+18.8) = +171.1
7.1 x (+21.2) = +150.5
9.3 x (+22.1) = +205.5
12.0 x (+23.1) = +277.2
13.4 x (+24.2) = +324.3
12.3 x (-3.1) = -38.1
+1128.6 -47.9
- 47.9
1080.70 sq ft

The double area, then, is 1080.70 sq ft; therefore, the area is one-half of that, or 540.35 sq. ft.

By similar method you can compute sta. 306 + 00, we have 408.40 sq. ft. Therefore, total earth to be removed (or cut) from sta. 305 + 00 to sta. 306 + 00, is

$$\left(\frac{540.35 + 408.40}{2}\right) \times \frac{100}{27},$$

or 1756.90 cu. yds.

LAYOUT/STAKEOUT PROCEDURES

The design survey is followed by the construction survey, which consists broadly of (1) the "layout" or "stakeout" survey, and (2) the "as-built" survey.

In a layout survey, horizontal and/or vertical control points are located and marked (that is, "staked out") for the guidance of those who will do the actual construction work. Details of a highway stakeout will vary with the type of highway being built and the type of equipment used to build it.

The final alignment as shown on the approved plans is staked out in the field by transit-tape survey. The control is the P-line or survey base line established in the design data survey. Curves are staked out and the final stationing is established. Sufficient control points are referenced to permit reestablishment at any time of the centerline.

On tangents (meaning straight-line stretches with neither horizontal nor vertical curvature), reference hubs or stakes are usually set on one side only of the centerline, at 100-ft (or perhaps, on a wide highway, at 50-ft) intervals. On horizontal and/or vertical curves they are set at closer intervals, and on a horizontal curve usually on both sides of the centerline. On a small-radius street-corner curve a hub might be set at the center of the circle of which the curve is a part, so that the construction workers may outline the curve by swinging the radius with the tape.

Rough Grading

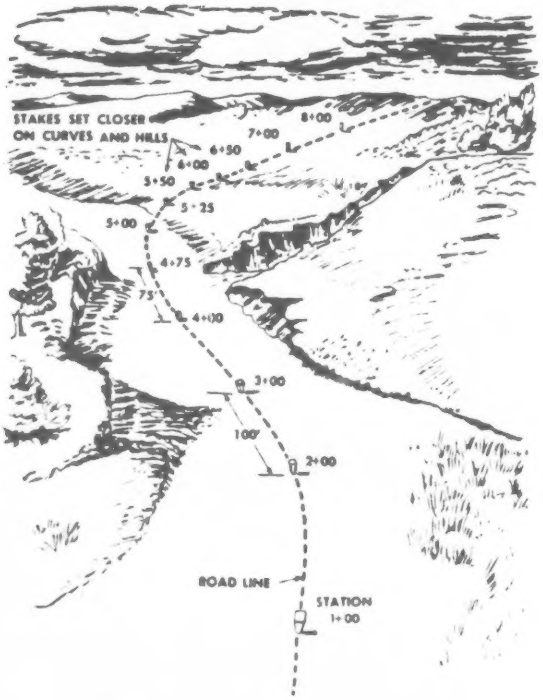
The first major step in highway construction is usually the "rough grading"—that is, the earthmoving which is required to bring the surface up to, or down to, approximately the elevation prescribed for the "subgrade". The subgrade is the surface of natural soil on which the pavement will be laid. Subgrade elevation therefore equals grade (finished surface) elevation minus the thickness of the pavement.

In rough grading the equipment operators are usually guided by grade stakes, set along the centerline, and each marked with the vertical depth of "cut" or "fill" required to bring the surface to grade elevation. The surveyor must indicate the station markings and the "cut" and "fill" directions on stakes. Let's look at the stakes on the centerline of the road-building job. The starting point is the first station in the survey; this station is numbered 0 + 00. The next station is normally 100 ft farther and is marked 1 + 00; the third station is another hundred feet farther and is marked 2 + 00; and so on. On sharp curves on rough ground the stakes may be closer together. (See fig. 16-21.) Generally the station markings face the starting point. The mark ℄, which is also on the side facing the starting point, is used to indicate that the stake is a centerline stake.

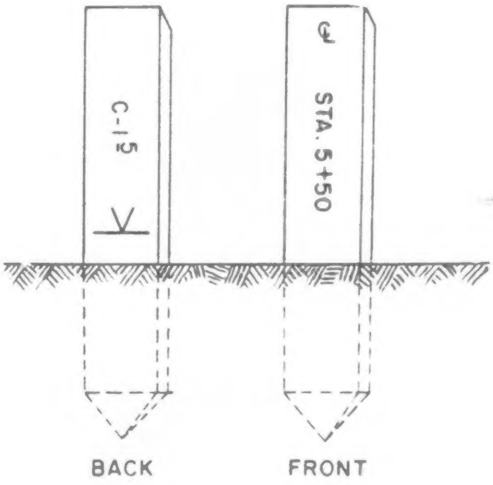
A "cut" is designated by the letter C, and the "fill" is indicated by the letter F. Numerals follow the letters to indicate the amount that the ground should be cut or filled. The symbol $C\text{-}1^{\underline{5}}$ indicates that the existing ground be cut 1.5 ft as measured from the reference mark. [During rough grading, the cut and fill are generally just carried up to the nearest half foot—exact grade elevations are later marked with hubs (blue tops).] This mark ($\underline{\vee}$) is called a "crowfoot". The appex of the "V" indicates the direction of the required change in elevation; so that a cut is indicated by "$\underline{\vee}$" and a fill is indicated by "$\overline{\wedge}$". Some surveyors generally mark the grade stake only with a negative or a positive number and the crowfoot indicating the cut or fill respectively. In this training manual however, the letters "C" and "F" are included for more emphasis for the students. Figure 16-22 shows a cut stake which also happens to be a centerline marker. You see that the station mark is written on the front of the stake and on the back the construction information. On other grade stakes other than the centerline stakes, the construction information should be on the front and the stationing are written on the back.

The stake shown in figure 16-23 indicates that fill operations are to be performed. The letter "F" at the top of the stake stands for "fill". The numerals $2^{\underline{0}}$ indicate that 2 feet of fill are required to bring the construction up to grade.

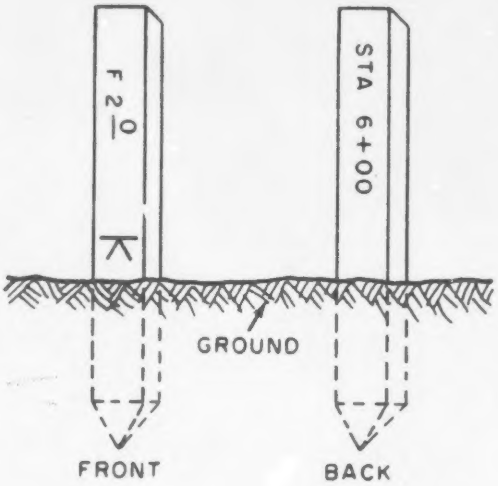
Some grade stakes indicate that no cutting or filling is required. Figure 16-24 for example,



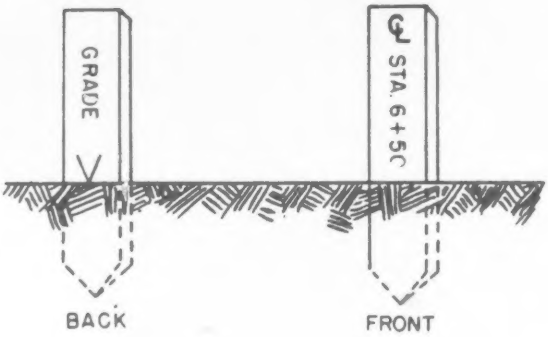
115.67
Figure 16-21. —Station markings.



29.79
Figure 16-22. —Cut stake.

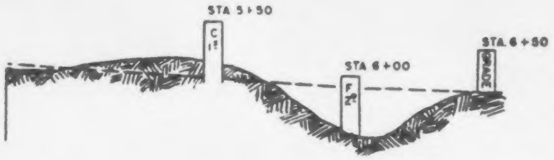


29.80(45B)
Figure 16-23.—Fill stake (not centerline stake).



29.81
Figure 16-24.—Stake on proper grade.

shows a grade stake that is on the proper grade and also a centerline stake. The word "GRADE" (or GRD) is on the back of the stake, and the crowfoot mark may not be indicated; some surveyors prefer to use crowfoot mark in all grade stakes. If this grade stake is not a centerline stake the "GRD" mark will be written on the front of the stake. If the three stakes shown in figures 16-22, 16-23, and 16-24 were on the centerline of the same survey construction job—a road, for instance—the shape of the land to be graded might look like figure 16-25.



115.68
Figure 16-25.—A profile of land.

Finish Grading

When you do final grading, you are likely to work with stakes called "BLUE TOPS". These are stakes driven into the ground until the top is at the exact elevation of the finished grade, as determined by the surveying crew. When the top of the stake is at the desired finish grade elevation, it is colored with blue lumber crayon (keel) to identify it as a finished grade stake. (Other colors may be used, but blue is the usual color.) This procedure is explained in the next section.

A common procedure is to set a line of hubs—offset, when feasible, to avoid displacement during construction work. Beside each hub a "guard stake" is driven, on which the data relating to the hub is inscribed. The guard stake usually shows the station of the hub, and the elevation of the top of same. The elevation and station markings may be required only at station points, otherwise, all that is needed is the blue top and the guard stake with flagging.

Setting Grade Stakes

Grade stakes are usually set after the centerline has been laid out and marked with hubs and guard stakes. They can be reestablished if the markers are disturbed. Elevations are usually determined by engineer's level and level rod. One procedure you can use for setting grade stakes is as follows.

a. From bench marks, turn levels on the centerline hubs, or on the ground next to a grade stake, at each station.

b. Reduce the notes so as to obtain hub-top or ground elevation.

c. Obtain the finished grade elevation for each station from the construction plans.

d. Compute the difference between finished grade and the hub or ground elevation to determine the cut or fill at each station.

e. Go back down the line and mark the cut or fill on each grade stake or guard stake.

The elevations and the cuts or fills may be recorded in the level notes, or they may be set down on a "construction sheet" as explained later in this chapter.

Another procedure may be used which combines the above operations so that computations may be completed while at each station, and the cut or fill marked on the stake at once. As before, levels are run from BM's; the procedure at each station is as follows:

a. Determine the ground elevation of station from the level notes to obtain HI.

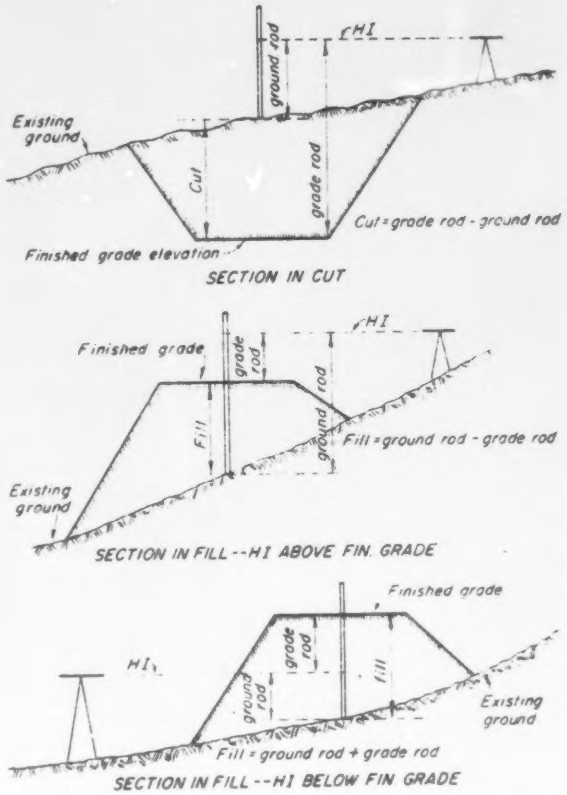
b. Obtain finished grade for the station from the plans.

c. Compute the difference between the HI and finished grade; this vertical distance is called grade rod.

d. Read a rod held on the hub top or ground point for which the cut or fill is desired. This rod reading is called "ground rod".

e. Determine the cut or fill by adding or subtracting grade rod and ground rod, according to the circumstances, as shown in figure 16-26.

f. Mark the cut or fill on the stake.



45.389

Figure 16-26.—Computing cut or fill from grade rod and ground rod.

Still another procedure involves the use of "blue tops". This procedure lends itself primarily to final grading operations. It is carried out as follows:

a. Study construction plans and centerline profiles to determine for each station (1) the exact profile elevation, and (2) the horizontal distance from centerline to the edge of the shoulder.

b. At each station measure the horizontal distance from centerline to shoulder edge and drive a grade stake at this point on each side. Sometimes it is advisable to offset these stakes a few feet to avoid displacement during construction.

c. Set the top of the stake even with the grade elevation, using level and rod. This is accomplished by measuring down from the HI a distance equal to the grade rod (determined by subtracting grade elevation from the HI). The target on the rod is set at the grade-rod reading; the rod is held on the top of the stake; and after a few trials, the stake is driven into the ground until the horizontal hair of the level intersects the rod level indicated by the target. Color the top of a stake with blue crayon (keel).

d. Where the tops of stakes cannot be set to grade because grade elevation is too far below or above the ground line, ordinary grade stakes marked with the cut or fill are set as in rough grading. However, for final grading it is usually possible to get a good many "blue tops".

Where grade stakes cannot be driven, as in hard coral or rock areas, you must use ingenuity to set and preserve grade markings in a variety of conditions. Markings may often be made on rock itself with a chisel or with keel.

Setting Slope Stakes

Slope stakes are set at the intersection of the planned slope with the original ground; they indicate the earthwork limits on each side of the centerline. Minimum areas to be cleared and grubbed extend outward about 6 feet from the slope stakes.

Figure 16-27 shows the approximate arrangement of various stakes and hubs as they appear in a final grading of a typical highway. Take a close look at the position of the slope stakes. The horizontal distance of a slope stake from the centerline varies, and to determine what it is you must know three things:

1. The width of the roadbed, including widths of shoulders and ditches, if any.
2. The side slope ratio (expressed in units of horizontal run in feet per foot of vertical rise or fall).
3. The difference in elevation between grade for the highway and the point on the natural ground line where the slope stake will be set.

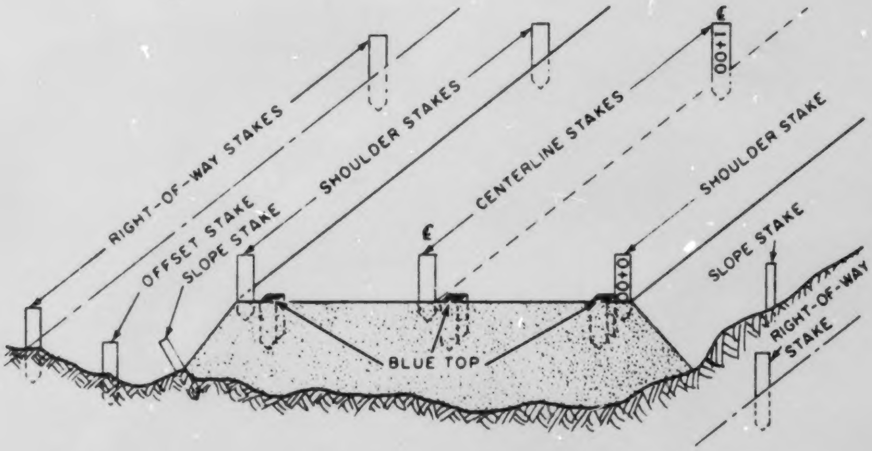
In figure 16-28A, d is the horizontal distance from the centerline to the slope stake, $\frac{W}{2}$ is the horizontal distance from the centerline to the top of the slope, h is the difference in elevation between finished grade and the ground at the slope stake, and s is the slope ratio. The product of h x s gives the run of the slope—that is, the horizontal distance the slope covers. The horizontal distance (d) of the slope stake from the centerline, then, equals the sum of $\frac{W}{2}$ plus hs.

For example: suppose that $\frac{W}{2}$ is 20 ft, h is 10 ft, and the bank is a 4:1 slope. Then hs is 10 x 4, or 40 ft, and d equals 20 ft + 40 ft, or 60 ft.

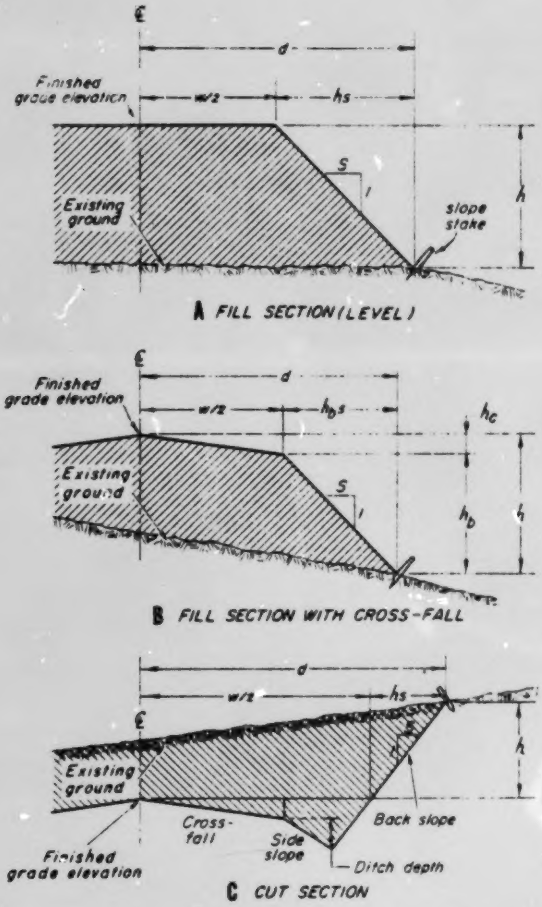
In practice you may have to take other factors into account, such as transverse slope or cross-fall of the pavement (sometimes called the "crown"), ditches, and so on. In figure 16-28B, for example, there is a cross-fall (h_c) across $\frac{W}{2}$, so that the run (horizontal distance covered) of the bank ($h_b s$) is the product of s x h_b instead of hs as in figure 16-28A. The cross-fall is usually constant, and may be obtained from the typical design section shown on the plans.

Figure 16-28C shows a cut section in which $\frac{W}{2}$ varies with cross-fall, side slope, ditch depth, and back slope. For example: assume that the distance from the centerline to the beginning of the side slope is 20 ft, that the cross-fall totals 1 ft, that the ditch depth is 1.5 ft, and that both the side slope and back slope ratios are 2:1. The distance $\frac{W}{2}$, then, comprises horizontal segments as follows:

a. From centerline to top of slope, which is 20 ft.



115.69(45B)
Figure 16-27.—Isometric view of the stake placement.



45.390
Figure 16-28.—Computing cut and fill sections for the proposed roadway.

b. Thence to ditch flow line, which equals the product of slope ratio (2) times ditch depth (1.5), or 3 ft.

c. Thence to the point on the back slope which is level with the finished centerline, which equals slope ratio (2) times difference in elevation i.e., cross-fall plus ditch depth, [2 (1 + 1.5)], or 5 ft.

The total distance $\frac{W}{2}$ in this case, then, is the sum of 20 + 3 + 5, or 28 ft.

Slope-Stake Party Procedure

Slope stakes are usually set with engineer's level, level rod, and metallic tape. In rough terrain, a hand level is generally used instead of an engineer's level. If the engineer's level is used, three men are generally employed for fieldwork, composed of: the instrumentman, the rodman, and one to hold the zero end of the tape at the centerline. When a hand level is used, two men can take care of the job—the instrumentman also holds the zero end of the tape and positions himself at the centerline station as he takes the rod reading. The procedure followed is a trial-and-error process. Under field conditions, the rodman is at times as much as 200 or 300 ft away from the instrumentman. If power equipment is operating nearby, or a wind is blowing, he cannot give the rodman oral instructions as to where to take trial shots—in fact, it is often the case that he does not have a clear view of the ground slope at the station being worked.

Consequently, the rodman must know as much as the instrumentman does about the theory and practice of setting slope stakes. The speed and efficiency of the party depend on the rodman more than on any other member. He must be constantly mentally alert.

The most practical field procedure requires that the rodman know the value of $\frac{W}{2}$ and s (the slope ratio). This is not difficult, since these values are usually constant for several stations, and the rodman can be informed when they change. A typical procedure for setting slope stakes is as follows:

a. Instrumentman computes centerline cut or fill, using HI, finished grade, and existing ground elevation (refer back to fig. 16-26).

b. Instrumentman calls or signals the centerline cut or fill to rodman.

c. Rodman mentally computes approximate value of d, by multiplying h x s and adding $\frac{W}{2}$. He pulls the tape taut while holding the tape at the computed distances out.

d. Noting the approximate rise or fall of the ground, rodman adjusts approximate value of d, moves to the d point, and sets up the rod for a trial shot.

e. Instrumentman quickly calculates cut or fill at this point and calls value to rodman.

f. Rodman compares this with his estimated cut or fill. He should be fairly close, and know at once whether to move toward or away from the centerline. Having a much shorter distance over which to estimate ground slope, he again estimates new cut or fill and hs + $\frac{W}{2}$, and moves rod to new d value.

g. Instrumentman again gives cut or fill, and if value checks, rodman calls or signals back cut or fill and distance.

h. Instrumentman quickly checks the two values mentally, and if correct, records values in field book, signaling "GOOD" to rodman.

i. Rodman marks and drives stake.

With practice, a good rodman, on fairly smooth ground, will seldom miss the first trial by more than 0.2 ft vertically, and will quite often hit the correct value on the first trial.

Figure 16-29 illustrates the application of these procedure to an actual situation. With regard to this problem, the following data are known:

The station is 15 + 00.

The $\frac{W}{2}$ (from the typical design section) is 20 ft.

The slope ratio is 1:1; therefore, s = 1.

Existing ground elevation at the centerline (from the previously run profile) is 364.00 ft.

The HI is determined to be 369.30 ft, at that setup.

The steps taken by instrumentman and rodman are as follows:

a. Instrumentman determines centerline cut by subtracting 350.7' from 364.0' to get cut 13.3'.

b. Rodman holds at centerline for a check. Rod should read 369.3 (the H.I.) minus 364.0, or 5.3 ft.

c. Instrumentman calls to rodman, "Cut 13.3."

d. Rodman computes d = 20 + (1 x 13.3) = 33.3 as he walks to the left.

e. As he approaches about 30.0 ft from the centerline, he estimates that the ground has fallen 4 ft. Therefore, he computes the new cut as 13.3 - 4.0, or 9.3 ft. This means a new d of 20 + (1 x 9.3) = 29.3 ft.

f. Rodman sets up the rod 29.3 ft from the centerline, as measured by metallic tape.

g. Instrumentman reads 10.1 on the rod, and computes new cut as 369.3 - (350.7 + 10.1), or 8.5 ft. (Note: Here you can also use the grade rod and ground rod values as explained earlier; the new cut then will be 18.6 - 10.1 = 8.5 ft; refer back to figure 16-26).

h. Instrumentman calls, "Cut 8.5" to rodman.

i. Rodman computes d = 20 + (1 x 8.5) = 28.5. He knows, therefore, that 29.3 ft from the centerline is too far out.

j. Figuring that the ground rises about 0.1 ft between 29.3 left and 28.5 left, the rodman calculates that the more nearly correct cut will be 8.5 + 0.1, or 8.6 ft.

k. Using this cut, rodman calculates new d as 20 + (1 x 8.6), and sets rod at 28.6' left.

l. Instrumentman reads 10.0 on the rod, and computes new cut as 369.3 - (350.7 + 10.0) = 8.6 ft.

m. Instrumentman calls, "Cut 8.6" to rodman.

n. Rodman sees that the actual cut of 8.6 ft agrees with his estimates cut of the same, and calls, "Cut 8.6 at 28.6" to the instrumentman.

o. Instrumentman checks d = 20 + (1 x 8.6) = 28.6, signals rodman, "GOOD", and enters $\frac{C\ 8^{\underline{6}}}{28^{\underline{6}}}$ in the field book.

p. Rodman marks a stake with "15 + 00" and "C $8^{\underline{6}}$", and drives it in the ground at 28.6 ft left.

More often, slope stakes may be set by using a hand level. Their distances out are generally measured to the nearest half or tenth of a foot. If a slope stake is placed in an offset position, the offset distance is also marked on the stake in order not to confuse the Equipment Operator as regards its actual location. Slope stakes are seldom used in areas requiring less than 2 feet cut or fill.

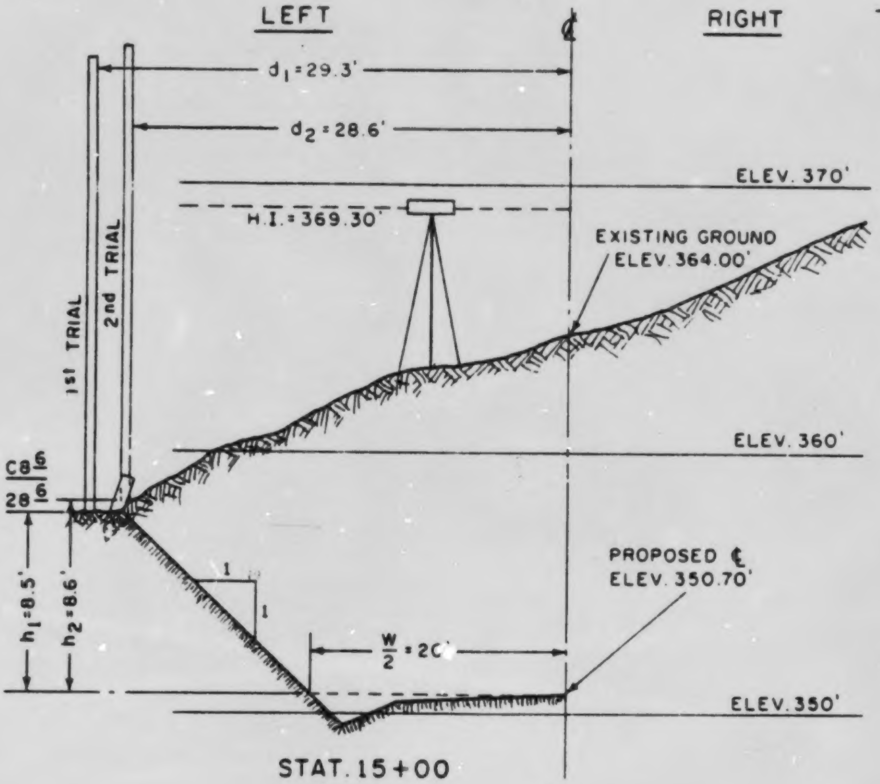
Curb and Gutter Stakeout

For a thoroughfare which will have a curb and gutter, these are usually constructed before the finish grading is done. The curb constructors obtain their line and grade from offset hubs like those described previously. Guided by these, the earthmovers make the excavation for the curb, the formsetters set the forms, and the concrete men pour, finish, and cure the curb.

Once the curb has been constructed, shaping the subgrade to correct subgrade elevation and laying the pavement to correct finish grade is simply a matter of measuring down the correct distance from a cord stretched from the top of one curb to the top of the curb opposite.

Pavement Stakeout

Pavement stakeout will depend on the type of paving equipment used. Steps in the method



45.802

Figure 16-29.—Slope stakes stakeout.

commonly used for paving concrete highways are (1) setting a double line of steel side forms, equipped with flanges which serve as tracks for traveling paving equipment, (2) filling the space between the forms with concrete poured from a concrete paving machine (commonly called just a paver), (3) spreading the concrete with a mechanical spreader which travels on the flanges of the side forms, and (4) finishing the surface with a finisher, a machine which also travels on the side forms.

The line-and-grade problem (that is, the layout or stakeout problem) here consists principally of setting the side forms to correct line, with the upper edges of the flanges at the grade prescribed for the highway. If the finished grade shown on the plans is centerline grade, then the forms are set with tops at centerline grade less cross-fall. If the design elevations are shown for points other than those on the centerline, the form elevation is related to the design points as indicated by the typical section.

Stakeout may be accomplished by setting a line or lines of offset hubs, as previously described. Sometimes, however, a line of hubs is driven along the line the forms will occupy and driven to grade elevation less the depth of a side form. The forms are then set to line and grade by simply placing them on the hubs.

Concrete paving is also done by the slip form method, in which, instead of a complete double line of forms, a sliding or traveling section of formwork is an integral part of the spreading and finishing machinery. The machinery is kept on line, and the pavement finished at grade, by a control device or devices. The line control device usually follows a wire stretched between rods which are offset from the pavement edge.

Forms are not usually used in asphalt paving. Asphalt paving equipment, in general, is designed

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to lay the pavement at a given thickness, following the fine-graded subgrade surface. The manner in which a given piece of equipment is kept on line varies, and the stakeout for equipment varies accordingly.

UTILITIES SURVEYS

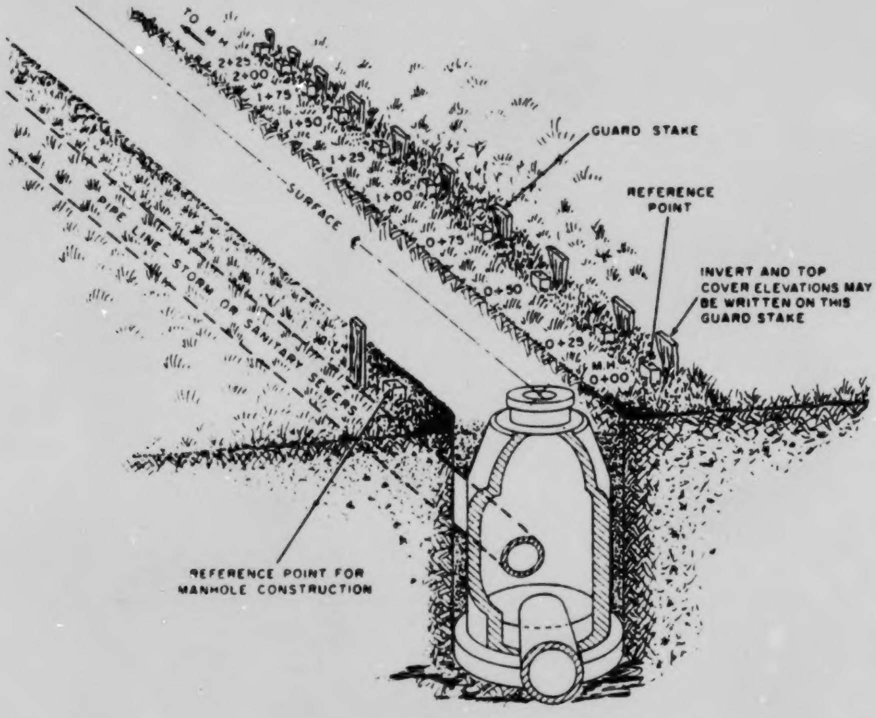
"Utilities" is a general term applied to pipe lines (such as sewer, water, gas, and oil pipe lines), communications lines (such as telephone or telegraph lines), and electric power lines. For an above-ground utility (such as a pole-mounted telephone, telegraph, or power line), the survey problem consists simply of locating the line horizontally as prescribed and marking the stations where poles or towers are to be erected. Often the directions of guys and anchors may be staked as well, and sometimes pole height for vertical clearance of obstructions is determined.

UNDERGROUND UTILITIES

For an underground utility it is often necessary to determine both line and grade. For pressure lines (such as water lines) it is usually necessary to stake out line only, since the only grade requirement is that prescribed depth of soil cover be maintained. However, it may be necessary to stake elevations for pressure lines being installed in an area which is to be graded downward, or in which other, conflicting underground utilities will be constructed.

Gravity flow lines, such as storm sewer lines, require staking for grade to ensure that pipe is installed at the design elevation and at the gradient (slope) designed to ensure gravity flow through the pipe.

Grade for an underground pipe is given in terms of the elevation of the "invert", the invert being the line of points along the lowest part of the inner surface of the pipe. Figure 16-30 illustrates a common method of staking out an underground sewer pipe. Both alignment and


TO MH
2+25
2+00
1+75
1+50
1+25
1+00
0+75
0+50
0+25
MH
0+00
SURFACE
PIPE LINE - STORM OR SANITARY SEWERS
GUARD STAKE
REFERENCE POINT
INVERT AND TOP COVER ELEVATIONS MAY BE WRITTEN ON THIS GUARD STAKE
REFERENCE POINT FOR MANHOLE CONSTRUCTION

45.392(45B)
Figure 16-30.—Sewerline stakeout.

elevation are facilitated by a line of offset hubs, set at 25-ft intervals. Adjacent to each hub there is a guard stake, on which the station of each hub is marked. The elevation of the top of the hub is also given, and the vertical distance from the top of the hub to grade for the invert of the pipe as well.

The horizontal alignment of the hubs gives the earthmovers their line; they dig the pipe trench the offset distance away from the line of hubs. The cut distance marked on each guard stake tells you how far below the top of the hub the trench should be cut.

ABOVE GROUND UTILITIES

Above-ground utilities are usually electrical power or telephone or telegraph communication lines strung on poles or towers. The location of the distribution (service) part of one of these is usually controlled by the locations of the buildings it serves — meaning that the service or distribution line will usually follow the streets on which the buildings are located. For the transmission part, however, judgment in the selection of a route is usually required. By transmission part we mean that part of a line which carries (for example) high-voltage electrical power from the power plant to points from which the power is distributed to consumer outlets.

STRUCTURAL SURVEYS

By a "structural" survey we mean one which is part of the chain of human activities which will bring a structure (such as a building, a bridge, or a pier) into existence.

EARTHWORK

As is the case with a highway, the first major step in the construction of a structure is usually the rough grading — that is, the earthmoving required to bring the surface of the site up to or down to the approximate specified rough grade.

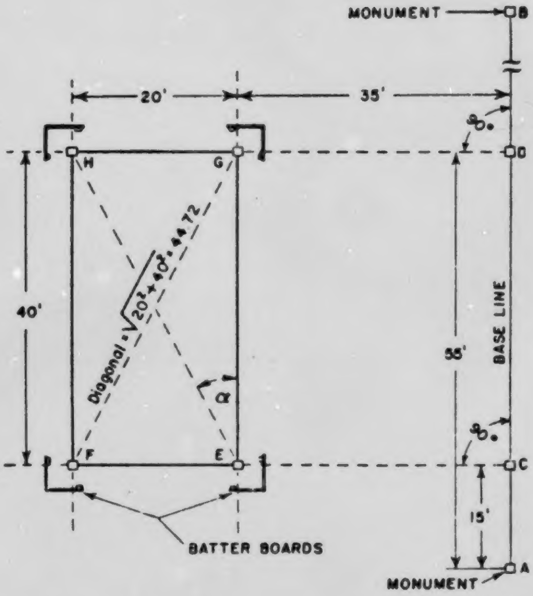
The stakeout for rough grading is commonly done by the "grid" method. The area to be graded (which is shown, together with prescribed finish grade elevations, on the site or plot plan) is laid off in 25-, 50-, or 100-ft grid squares. The elevation at each corner point is determined, the difference between that and prescribed grade elevation is computed, and a grade stake, marked with the depth of cut or fill, is driven at the point.

BUILDING STAKEOUT

If the structure is a building, the next major step after the rough grading is the building stakeout — that is, the locating and staking of the main horizontal control points of the building. These are usually the principal corner points, plus any other points of intersection between building lines.

The procedure followed varies with circumstances. Figure 16-31 illustrates a simple stakeout situation. In this case the site plan shows that the building is to be a 40-ft x 20-ft rectangular structure, located with one of the long sides parallel to and 35 ft away from a base line. The base line is indicated at the site and on the plans by monuments A and B.

One of the short sides of the building is to lie on a line running from C, a point on AB 15 ft from A, perpendicular to AB. The other short side is to lie on a similar line running from D, a point on AB 40 ft from C and therefore 40 + 15, or 55 ft from A.



45.393

Figure 16-31. — Building stakeout.

The steps in the stakeout procedure would be about as follows:

1. Set up transit at monument A, train telescope on a marker held on monument B, and have hubs driven on the line of sight, one at C 15 ft from A, and the other at D 55 ft from A and 40 ft from C.
2. Shift the transit to C, train on B, match the zeros, and turn 90°L. Measure off 35 ft from C on the line of sight and drive a stake to locate E. Measure off 55 ft from C (or 20 ft from E) and drive another stake to locate F.
3. Shift the transit to D and repeat the procedure described in (2) to locate and stake points G and H.

The accuracy of a rectangular stakeout can be checked by measuring the diagonals of the rectangle. The diagonals should, of course, be equal; and you can determine what the correct length of each diagonal should be by applying the Pythagorean theorem as shown in figure 16-31.

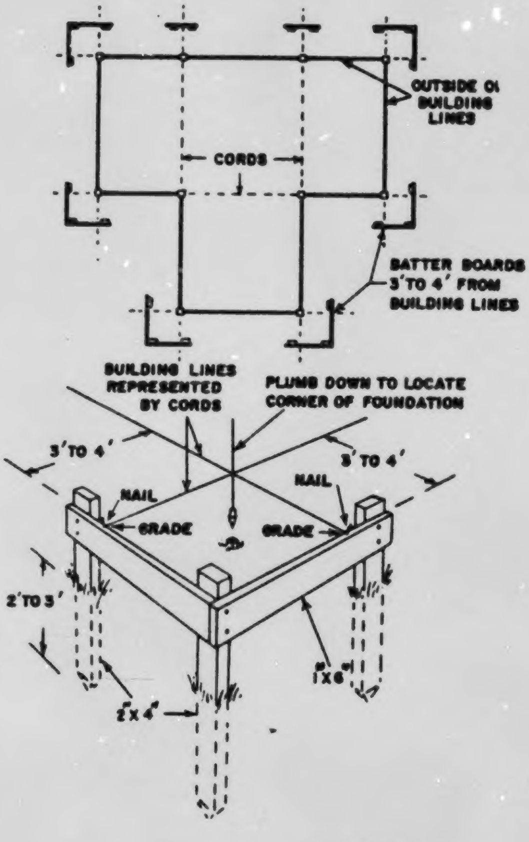
For a large rectangle it may be more convenient to check the accuracy of the stakeout by angular measurement with the transit. For example: you can determine the correct size of angle GEH, (let's call α) in figure 16-31 by any convenient right-triangle solution—such as $\tan \alpha = \frac{20}{40} = 0.50000$. The angle with tangent 0.50000 measures (to the nearest minute) 26° 34'. This being the case, angle FEH should measure 90° 00' - 26° 34', or 63° 26'. The corresponding angles at the other three corners should have the same dimensions. If the sizes as actually measured vary at any corner, the stakeout is inaccurate.

It may be necessary to repeat angles to obtain an angular precision appropriate to the lengths of the lines being checked.

BATTER BOARDS

The EA locates the corners of the building and establishes a Temporary Bench Mark (T.B.M.) from the known Bench Mark (B.M.). This T.B.M. is located near the construction area in such a way that it can be used as a reference from time to time. To mark the general location of the structure, stakes or slats are set. These will guide the initial excavation and rough grading. However, the stakes will be disturbed or destroyed during the site preparation and some more suitable marks must be placed to continue the construction work. These suitable marks are called BATTER BOARDS. They are more or less temporary devices which support stretched cords that mark the outline and grade of the structure.

Batter boards consist of 2" x 4" stakes driven into the ground and a crosspiece of 1" x 6" lumber nailed to each stake. The stakes are driven about 3 to 4 feet away from the building line where they will not be disturbed by the construction. They are driven far enough apart to straddle the line to be marked. Note in figure 16-32, only three stakes are driven on outside corners, one of them being a common post for two directions. The length of the stakes is determined by the required grade line. They


OUTSIDE OF BUILDING LINES
CORDS
BATTER BOARDS 3' TO 4' FROM BUILDING LINES
BUILDING LINES REPRESENTED BY CORDS
PLUMB DOWN TO LOCATE CORNER OF FOUNDATION
3' TO 4'
3' TO 4'
NAIL
NAIL
GRADE
GRADE
2' TO 3'
2"x4"
1"x6"

45.394
Figure 16-32.—Batter boards.

must be long enough to accept the 1" x 6" crosspiece to mark the grade. The 1" x 6" crosspiece is cut long enough to join both stakes and is nailed firmly to them after the grade has been established. The top of the crosspiece becomes the mark from which the grade will be measured. All batter boards for one structure are set to the same grade or level line. A transit is used to locate the building lines and to mark them on the top edge of the crosspiece. A nail is driven at each of these marked points or a "V" notch is carved at the top outer edge of the crosspiece towards the marked point and the nail is driven on the outer face of the board. When a string is stretched over the top edge of the two batter boards and is held against the nails or against the bottom of the notch, the string will define the outside building line and grade elevation.

Sometimes a transit is not available for marking the building line on the batter boards, but the corner stakes have not been disturbed. By stretching a cord over two opposite batter boards and using plumb bobs held over the corner stakes, the building line can be transferred to the batter boards. The cord is moved on each batter board until it just touches both plumb bob strings. This position of the cords is marked and nails are driven.

Batter boards are set and marked as follows:

1. After the corner stakes are laid out, 2" x 4" stakes are driven 3 to 4 feet outside of each corner. These are selected to bring all crosspieces to the same elevation.
2. These stakes are marked at the grade of the top of the foundation, or at some whole number of inches, or feet, above or below the top of the foundation, using a level to mark the same grade or elevation on all stakes.
3. 1" x 6" boards are nailed to the stakes so the top edge of the boards is flush with the grade marks.
4. The prolongation of the building lines on the batter boards is located by using a transit or by using a line and plumb bob.
5. Nails are driven into the top edges of the batter boards (or notched) to mark the building line.

CULVERTS AND BRIDGES

As with other types of layout for construction, the stakeout of culverts and bridges generally includes providing line and grade. The procedures and precision required will vary with the magnitude and complexity of the job.

Ditches and Culverts

For minor open drains or outfall ditches a few feet deep, a single line of stakes will serve for both alignment and grade. By running profile levels, you can determine the elevations of the tops of the stakes. As a guide to the construction workers, mark the cut on each stake to show the depth of drain below each station.

For drains that are very deep, you will find it necessary to cross-section the line and set slope stakes. The grade for a ditch is measured along the "flow line"—that is, along the bottom of the ditch.

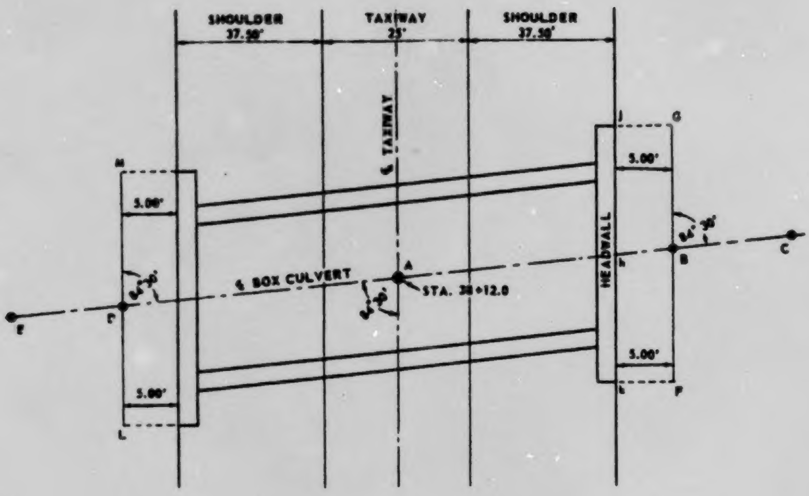
When staking pipe culverts without wing walls and aprons, the alignment and invert grade are all that is required. When head walls, wing walls, and aprons are used to intercept drainage water, to retain earthwork, and to prevent erosion, grade stakes as well as horizontal alignment stakes will be required. Large bridge-type culverts and box culverts require stakes and hubs for batter board alignment similar to that for a building layout.

Figure 16-33 illustrates the stakeout of a box culvert. The angle at which the culvert will cross below the roadway or taxiway may be inscribed on the plans, or it may be protracted from the plans.

Assume that this angle is 84° 30', as shown. To run the centerline of the culvert, set up the transit at A and turn the 84° 30' angle from the centerline of the taxiway.

Place reference stakes at B, C, D, and E, along the culvert centerline, far enough beyond the limits of the culvert to ensure that they are not disturbed by construction work. In this case, points B and D are set arbitrarily at 5 ft (measured at right angles) from the location of the outside face of the culvert headwall.

To facilitate the stakeout, point h may be staked. From h the locations of points j and k may be measured and staked, using for the distance one-half the length of the headwall as that length is shown on the plans. Set stakes at F and G, points directly opposite and on lines at right angles to the ends of the headwalls. Set stakes similarly at L and M. Set grade stakes near B and D for the invert or flow line of the culvert.



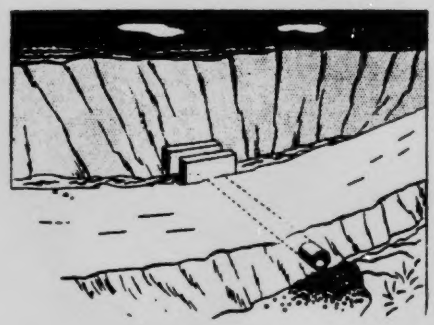
45.395
Figure 16-33.—Stakeout of a box culvert.

The stakes thus set are sufficient to locate the forms for the headwalls and for the barrel of the culvert. Figure 16-34 shows one of a number of types of pipe inlet and culvert—a type suitable for picking up side-surface drains adjacent to a landing strip or roadway embankment. Stakes for both horizontal alignment and elevations are required. Figure 16-35 shows the stakeout of a pipe culvert, wingwall, and apron.

Bridge Substructures

The substructure of a fixed bridge consists of the end and intermediate supports and their foundations. Bridge substructures are divided into two main types of supports: end supports called "ABUTMENTS" and intermediate supports called "BENTS" or "PIERS."

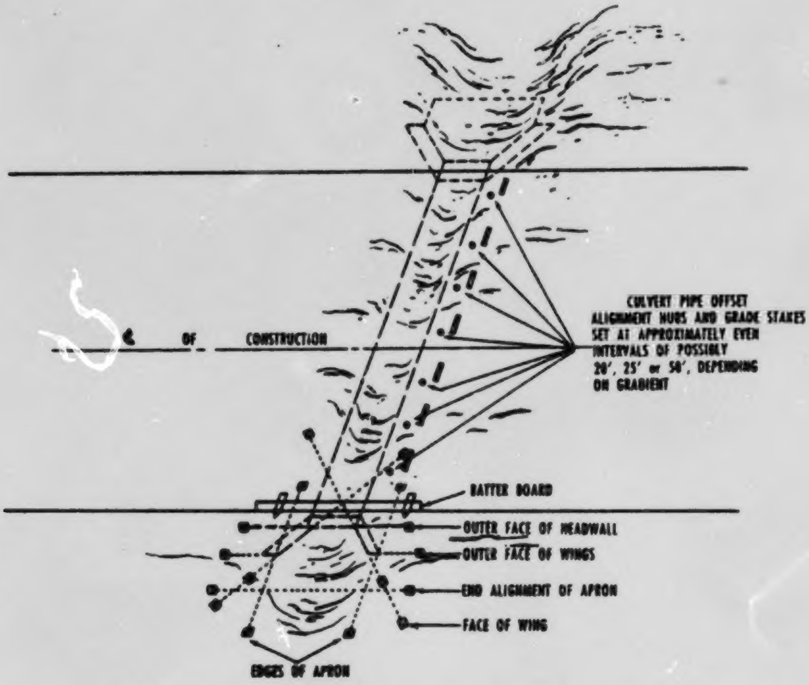
ABUTMENTS.—The ground support at each end of a bridge is called an abutment. Construction plans will show the details of the abutments. Check the layout after excavation and prior to pouring of concrete. You must check abutment elevations and, in the case of concrete, establish lines for setting forms. Abutments must be staked according to the construction plans, and distance between abutments must be within steel fabrication limits, especially for prefabricated sections. Abutment stakes should be tied to the horizontal control system to meet these accuracy requirements.



45.396
Figure 16-34.—Ditch inlet and pipe culvert.

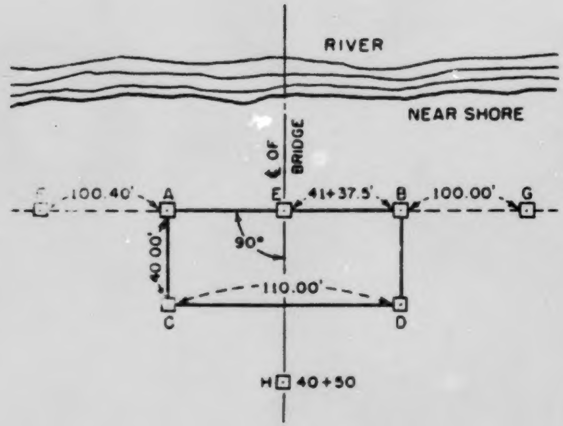
Following is a typical procedure for surveying an abutment which is to be at right angles to the centerline of the bridge. In figure 16-36 the foundation of a concrete abutment, ABDC, is shown in plan. AB is the face of the abutment foundation. Establish two convenient points H and J near the abutment CD, on the bridge centerline. Set a stake at E (station 41 + 37.50), the station designated on the plan for the abutment face.

Set up the transit at E, train on H, match the zeros, turn 90° angles and locate A and B


℄ OF CONSTRUCTION
CULVERT PIPE OFFSET ALIGNMENT HUBS AND GRADE STAKES SET AT APPROXIMATELY EVEN INTERVALS OF POSSIBLY 20', 25' or 50', DEPENDING ON GRADIENT
BATTER BOARD
OUTER FACE OF HEADWALL
OUTER FACE OF WINGS
END ALIGNMENT OF APRON
FACE OF WING
EDGES OF APRON

45.397

Figure 16-35.—Stakeout of a pipe culvert, wingwall and apron.



45.396

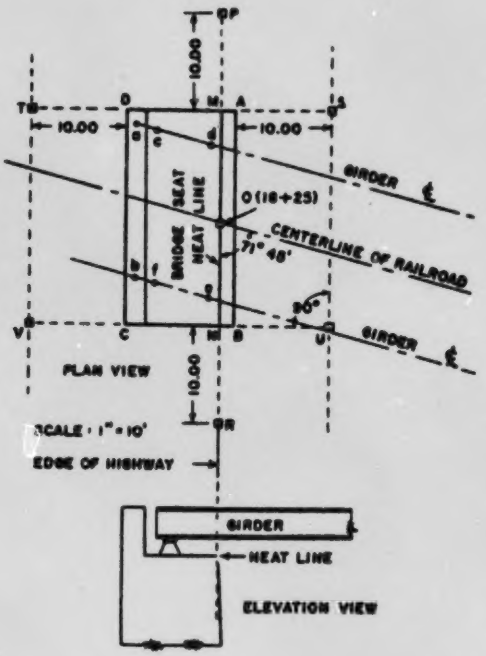
Figure 16-36.—Staking a right-angle abutment.

at the correct distance from E. Reference line AB by setting stakes at F and G at the indicated distances from A and B. Set temporary stakes at C and D to mark the other corners of the foundation.

Sometimes the alignment of a bridge is not at right angles to the centerline of the stream or road it crosses. In a case like this, the abutment is askew (other than a right angle) to the centerline of the stream or road. Slight modifications are necessary in staking out an askew abutment.

Figure 16-37 shows the plan for an askew near-side abutment of a railroad bridge over a highway. The outside line of the foundation is ABCD. The neat line of the face of the abutment is MN. Set stakes to define the direction of MN and ends AD and BC. The stakes P, S, U, R, V, and T are offset from the abutment so they will not be disturbed by foundation excavating. The general procedure is as follows:

a. Take the dimensions for setting necessary stakes from the abutment plans. Set the



45.399
Figure 16-37.—Staking an askew abutment.

temporary point 0 at the station location indicated.

b. With the instrument at 0, sight along the centerline of the railroad, turn the skew angle (71° 45'), and set the permanent stakes P and R, and set points M and N.

c. With the instrument at M, sight R, turn 90', and set permanent stakes S and T.

d. With the instrument at N, sight P, turn 90°, and set permanent stakes U and V.

The face of the abutment is defined by P and R. Stakes S, T, U, and V define the face of the end forms. When construction begins, set stakes at A, B, C, and D by measuring from the offset stakes. (These stakes are knocked out as the excavation progresses.)

Concrete for the foundation is poured into the excavation; if forms are needed for the foundation, measure distances from the offset stakes. Set the elevations of the top and bottom of the foundation from bench marks outside the excavation area.

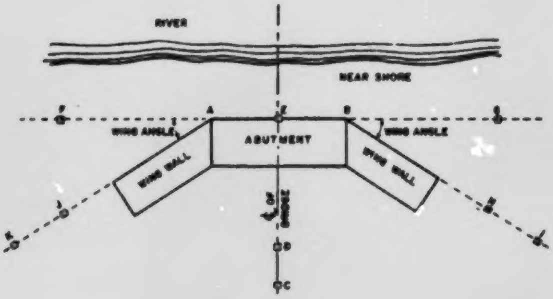
When the foundation has been poured to grade and has had a day to set, mark, temporary points on the top at M and N by measuring 10 ft plus the distance AM (BN) from the offset stakes S and U. Check forms by measuring the equal diagonals MC and ND. Mark points denoting elevation directly on the forms and give the data to the petty officer in charge of the construction.

After the bridge seat is poured, mark point 0. After the rear wall has been poured, mark points defining the girder centerlines: a, b, c, d, e, and f. These points will be used for the accurate location of the "bearing plates" which will support the girders.

ABUTMENT WING WALLS.—Figure 16-38 illustrates the stakeout of abutment wing walls. A typical procedure is as follows:

a. Set up the instrument at B, turn the wing angle from G; set reference stakes H and I; measure distances BH and BI. Set up at A and repeat this procedure to establish J and K. Use reference lines FG, BH, and AJ to set temporary stakes marking the corners of the excavation for the foundation. The procedure previously described for abutments is then followed. If abutment or wing-wall faces are battered (inclined rather than vertical), lines are established for both top and bottom.

b. To stake out wing walls for askew abutments to the centerline of a bridge, follow the procedure previously described for askew abutments. Set up the instrument over N (fig. 16-37); sight on R; turn the wing angles; set reference stakes to establish the wing line from N. Establish the wing line from M in the same manner.



45.400
Figure 16-38.—Staking out abutment wingwalls.

PIERS.—After the centerline of the bridge is established, locate the piers by chaining if possible. If chaining is impracticable, locate the piers by triangulation. Set stakes establishing the centerline on each side of the river. Lay out CD and EF approximately at right angles to the centerline as shown in figure 16-39. For well-proportioned triangles the length of the baselines should equal at least one-half CE. To locate piers at A and B, the following procedure may be used:

a. Establish baselines CD and EF and carefully reference them.

b. Measure the length of each baseline with a degree of accuracy commensurate with the required accuracy of the line CE.

c. Measure all angles of the triangles CDE and EFC.

d. Compute the distance CE from the triangle CDE and check against the same distance computed from triangle EFC. The difference in computed lengths must check within the prescribed limits of error.

e. Compute angles BDC, ADC, BFE, and AFE.

f. Draw a triangulation diagram, showing computed angles and distances and measured angles and distances.

g. Turn the computed angles BCD, ADC, BFE, and AFE.

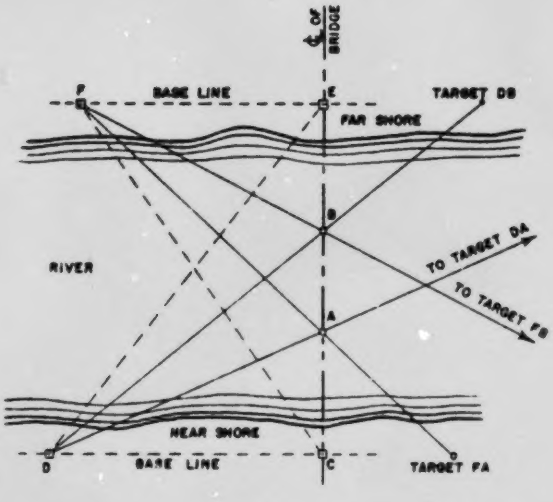
h. Set targets DA and DB on the far shore and FB and FA on the near shore, so that the intersecting lines can be reestablished without turning angles. Carefully reference these points.

i. Use two instruments to position piers. Occupy two points such as C and D simultaneously, using the intersection of sights CE and DA to locate the pier. Check the locations of points A and B within the limits of error by sighting along CE.

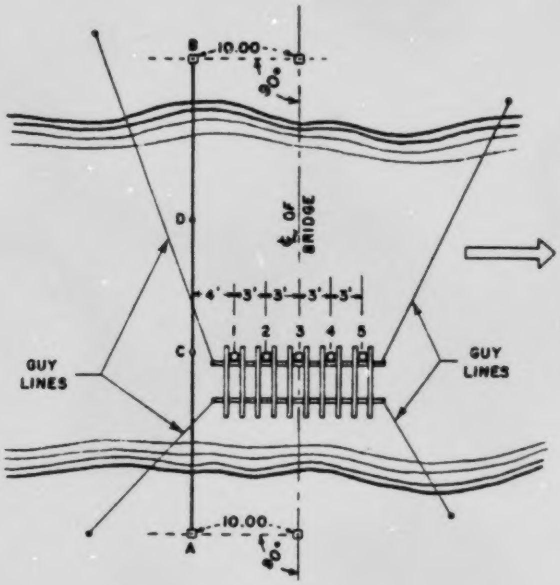
PILES. You may be required to position piles, record pile-driving data, and mark piles for cutoff. Figure 16-40 shows points A and B established as a reference line 10 ft from the centerline of a bridge. Stretch a wire rope between points A and B, with a piece of tape or a wire rope clip at each pile-bent position (such as C or D).

Locate the upstream pile (pile no. 1) by measuring an offset of 4 ft from the line AB at C. A template is then floated into position and nailed to pile No. 1 after it is driven. The rest of the piles are positioned by the template.

If it is impractical to stretch a wire rope to the far shore, set up a transit at a convenient distance from the centerline of the bridge, and position the piles by sighting on a mark located



45.401
Figure 16-39.—Method of locating piers.



45.402
Figure 16-40.—Method of positioning piles.

the same distance from the centerline of the template. Before driving piles, you must measure the length of piles. Measure the distance between piles by chaining.

During driving, keep a complete record of the following: location and number of piles, dimensions, kind of woods total penetration, average drop of hammer and average penetration under last five blows, penetration under last blow, and amount of cutoff. Mark elevations on the two end piles for nailing two 3-inch by 12-inch planks to guide the saw in cutting the piles to the specified height.

Bridge-Site Grade Stakes

Elevations are taken from bench marks set in or near the construction area. Consider permanency, accessibility, and convenience when setting bench marks. Set grade stakes for a bridge site in the same manner as the grade stakes on any route survey. Make sure that the senior petty officer in charge of the job has sufficient information so that he may understand exactly what method is being used to designate grade.

CONSTRUCTION SHEETS

Several construction situations have been mentioned in which line and grade for construction are obtained from a line (or perhaps from two lines) of offset hubs. A guard stake adjacent to one of these hubs usually gives the station and elevation of the hub, grade for the structure at that station, and the vertical distance between the top of the hub and grade, marked C or F as appropriate.

This information is often recorded on a construction sheet (familiarly known as a cut sheet) like the one shown in figure 16-41. One advantage of the use of cut sheets is the fact that the information applying to every hub is preserved in the event that guard stakes are accidentally displaced. Another advantage is the fact that reproductions of the cut sheet can be given to construction supervisors, so that a supervisor may have all the essential construction data in his personal possession at all times.

AS-BUILT SURVEYS

The design phase of a construction project concludes with the preparation of the "working drawings", reproductions of which will guide the construction surveyors and the men who do the actual work of construction.

It is not often the case that a finished structure corresponds exactly to the original plans in every detail. Unexpected, usually unforeseeable, difficulties often make variations from the plans necessary—or, occasionally, variations may occur by accident which are economically unfeasible to correct.

The purpose of an "as-built" survey is to record these variations. The as-built survey procedure should begin as soon as it becomes feasible—meaning that the actual horizontal and vertical locations of features in the completed structure should be determined as soon as the features are erected.

In some cases variations from the original plans are recorded on new tracings of the working drawings, on which as-built data is recorded in the place of the original design data when the two happen to differ. In other cases, reproductions of the original drawings are used, with variations recorded by crossing out the original design data and writing in the as-built data.

In either case, the term AS-BUILT, together with the date of revision, is written in near the title block.

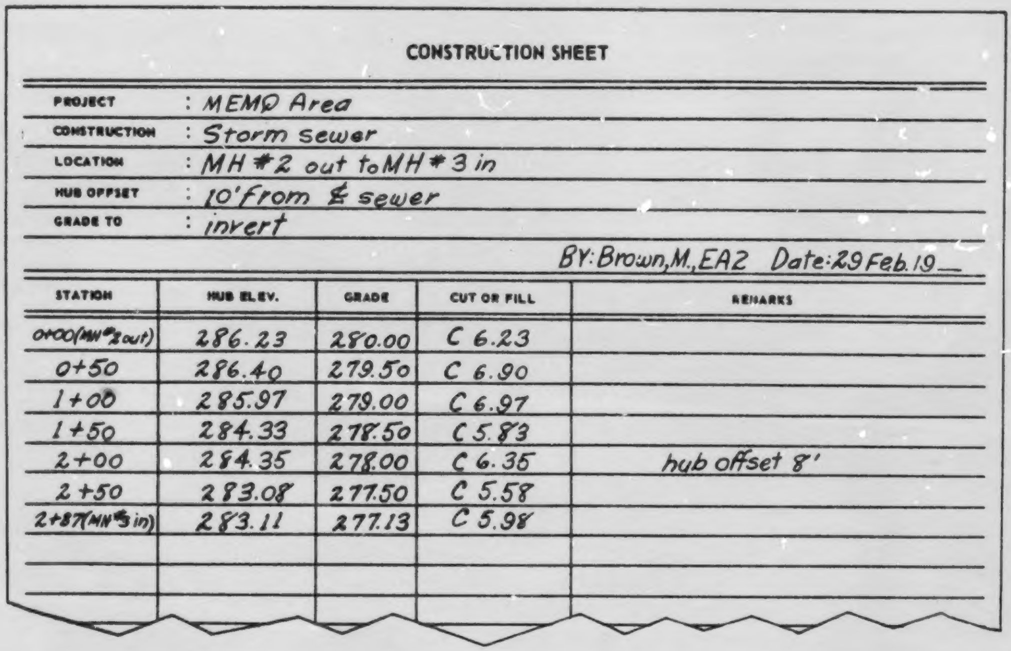
CONSTRUCTION SITE SAFETY

A survey party working at a construction site is always in a dangerous situation. Where blasting or logging is going on, inform the powder crew or logging crew of the location of the area in which surveyors are working. Also instruct the men of the survey party individually to be on the alert at all times—particularly to listen for the warning signal given by a powderman setting off a charge or a logger felling a tree.

When surveying near highways, railroads, or airstrips use red flagging generously, unless you are working in a combat area. Place flagging on the legs of your surveying equipment and at a few places along the tape. Put flags on rods and range poles. Attach some flagging to your hat and also to the back of your shirt or jacket.

Think constantly of personal safety when working near heavy construction equipment. Let the equipment operators know when surveyors are in the vicinity. Likewise, alert all members of the surveying crew, because an equipment operator's vision is often obscured by dust or by the equipment itself.

When ascending steep, rocky slopes, do not climb directly behind another man. If the man accidentally falls, or loosens a rock, or drops



CONSTRUCTION SHEET

PROJECT : MEMQ Area
CONSTRUCTION : Storm sewer
LOCATION : MH #2 out to MH #3 in
HUB OFFSET : 10' from ℄ sewer
GRADE TO : invert

BY: Brown, M., EA2 Date: 29 Feb. 19—

STATION	HUB ELEV.	GRADE	CUT OR FILL	REMARKS
0+00 (MH #2 out)	286.23	280.00	C 6.23	
0+50	286.40	279.50	C 6.90	
1+00	285.97	279.00	C 6.97	
1+50	284.33	278.50	C 5.83	
2+00	284.35	278.00	C 6.35	hub offset 8'
2+50	283.08	277.50	C 5.58	
2+87 (MH #3 in)	283.11	277.13	C 5.98	

45.403
Figure 16-41.—Typical construction sheet.

something, it might mean serious injury to anyone directly below him.

EXCAVATIONS

When your work involves excavation there are definite precautions to be observed to prevent accidents.

To avoid slides or cave-ins, the sides of excavations 5 ft or more deep, if they are steeper than the "angle of repose" (maximum angle at which material will "repose" without sliding), should be supported by substantial bracing, shoring, or sheet piling. Trenches in partly saturated or otherwise highly unstable soil should be stabilized with vertical sheet piling or suitable braces. Foundations of structures adjacent to excavations should be shored, braced, or underpinned as long as the excavation remains open. Excavated or other material should not be accumulated closer than two feet from the edge of an excavation. In a traffic area use barricades, safety signs, danger signals, red lights, or red flagging on at least two sides.

Do not enter a manhole until you are certain that is is free from dangerous gases. Do not guess. If there is any question at all as to whether a sewer is gas-free, wait for clearance from a competent authority. If necessary, provide first for thorough ventilation. Do not smoke in manholes, and if illumination is required, use only a safety flashlight or lantern.

Avoid contact with ALL electric wiring. Never throw a metal tape across electric wires—if you must chain across wiring, do it by breaking chain. Avoid placing yourself so that you might fall across wiring in the event of an accident.

When walking, always stay at least two feet away from the edge of a vertical excavation. Near thoroughfares or walkways, excavations should have temporary guardrails or barricades, and, if permissible, depending on combat conditions, red lights or torches should be kept alongside from sunset to sunrise.

TREE CLIMBING

Before climbing a tree, ascertain that is is safe to climb, and note well the condition of

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CHAPTER 6

CONSTRUCTION AND LAND SURVEYS

This chapter describes important factors involving route surveys, stakeout and as-built surveys, airfield surveys, waterfront surveys, earthwork computations, and land surveys. Construction surveys were discussed in *Engineering Aid 3&2*, and they are presented here from the viewpoint of the party chief.

Land surveying, as described in this chapter, is intended to acquaint the EA with procedures and legal aspects involved in this special type of survey. Land surveying includes reestablishing original land boundaries, establishing new boundaries, and preparation of legal property descriptions.

Technically speaking, construction surveying is engineering surveying. Its primary concern is the orderly process of obtaining data for the various phases of construction activities. As explained in *Engineering Aid 3 & 2*, construction surveying is divided into (1) the "layout" or "stakeout" survey, and (2) the "as-built" survey. The layout or stakeout survey includes the reconnaissance survey, the preliminary survey and the location survey. The as-built survey consists simply of determining horizontal and vertical locations of points as they were actually constructed.

The objectives of construction (or engineering) surveying include:

1. The obtaining of reconnaissance information and preliminary data required by engineers for selecting suitable routes and sites, and for preparing structural designs.
2. The defining of selected locations by establishing a system of reference points.
3. The guidance of construction forces by setting stakes or otherwise marking lines, grades, and principal points, and by giving technical assistance.
4. The measuring of construction items in place for the purpose of preparing progress reports.
5. The dimensioning of structures for preparation of as-built plans.

There are a great many different types of man-made structures; only a few of the most common can be discussed in this chapter. Those discussed are roads, highways, utilities (that is, sewer, power, gas, water, and fuel lines), airfields and waterfront structures. Building and bridge layout survey has been adequately covered in *Engineering Aid 3 & 2*.

For a structure which follows a specified route, the construction survey must usually be preceded by a ROUTE survey.

ROUTE SURVEYS

The term ROUTE, like many other terms, has more than one meaning, but the first definition given in most dictionaries is: "The course or way which is followed, or which is to be followed." A ROUTE SURVEY, then, is one which deals with the route (that is, the course or way) which a structure will follow.

Of the structures discussed in this section, two follow routes; these two are utilities lines and roads/highways. The route survey for a road or highway differs in important respects from that for a utilities line, and the survey for one type of utilities line may differ considerably from that for another. However, in general it may be said that the principal purposes of any route survey are:

1. To select one or more tentative GENERAL routes for the structure.

2. To gather enough information about the area covered by each general route to make it possible for designers to pinpoint the specific FINAL LOCATION of the route finally selected.
3. To mark this final location.

Consistent with these principal purposes, a route survey is usually broken down into three phases, as follows:

1. The RECONNAISSANCE survey (purpose 1).
2. The PRELIMINARY survey (purpose 2).
3. The FINAL LOCATION survey (purpose 3).

With regard to terminology, the route survey is not a part of, but is preliminary to, the construction survey. However, the two are interconnected to the extent that a discussion of construction surveying would be unintelligible without an understanding of the previous route survey.

The amount of detail involved in a route survey will vary, of course, in accordance with the type of structure and many other circumstances. Obviously, a route survey for a line of telephone or power poles requires less detail than a route survey for a superhighway. However, the primary purpose of any route survey is to select the route that will satisfy the requirements of the project with maximum economy and advantage.

HIGHWAY ROUTE SURVEY

Highway route surveys for construction involve considerations of curvatures, gradients, drainage, soil conditions, sight distance, safety, width, and roadside safeguards and surfaces to withstand the impact of traffic.

Reconnaissance Survey

The minimum information with which a highway reconnaissance party enters the field is the specified CONTROL POINTS—that is, the points which must be connected by the highway. If the highway is to follow the route of an already existing road, little—perhaps no—reconnaissance will be necessary. For a new highway, the entire area between the control points must be examined for possible routes.

A reconnaissance party usually functions under the direction of an officer acting as LOCATION ENGINEER; however, the enlisted party chief, in order to assist the location engineer intelligently, must know something about the general principles and practices of reconnaissance.

The first requirement is a mental picture of the general landscape of the area. Here, maps are of great importance—especially contour maps or composite aerial photographs (mosaic) of the surrounding area. After a thorough study of all available maps, the reconnaissance party should go over the ground—both by plane and by ground travel, if both are possible. The ground party will examine the natural features of the proposed route, see if the existing soil is stable for a normal roadway, and take note of approximate locations of available sources of construction materials (quarry sites and borrow pits) in the immediate vicinity.

Enough approximate elevations and distances are obtained to convey an idea of the grading, earthwork,. and other construction problems along the different routes. The locations and descriptions of streams, ridges, existing man-made features, and other obstacles are set down, as well as the locations of any objects which may offer advantages, such as existing bridges or low points on ridges.

Approximate methods are used for distance, direction, and elevation measurements. Distances may be scaled from available maps or, in the absence of maps, measured approximately by one of the approximate methods described in *Engineering Aid 3 & 2.* Elevations may be taken from contour maps or, in the absence of these, measured by altimeter or computed from vertical angles measured by clinometer. Directions (horizontal angles) may be measured by hand compass.

The reconnaissance report contains a summary of the information gathered. It usually also includes a description of alternative routes considered feasible by the location engineer, and a discussion of the controlling elements, economic considerations, and recommendations pertaining to each alternative route. All available maps, sketches and aerial photographs applying to the area are submitted with the report.

Preliminary Survey

A study of the reconnaissance report may indicate that only one of the suggested alternative routes is feasible, or the report may itself indicate only a single feasible route. If this is the case, the preliminary survey will be omitted and the next step will be the final location survey.

If alternatives exist, a narrow strip along each alternative route is surveyed and a PRELIMINARY MAP and a PROFILE of the strip are prepared. A transit party runs an open traverse approximately along the middle of the strip, setting stakes at every full station and setting hubs and stakes wherever the traverse changes direction. A level party follows the transit party, establishing bench marks and taking profile elevations along the traverse. The level party may also take cross-section elevation, or these may be left for the topographic party.

Finally, a topographic survey party locates all relevant details on the strip, such as buildings, property lines, streams, fences, bridges, and any other features, either natural or artificial, which may influence the selection of the final location.

If the route runs through wooded country, the traverse must usually be run by transit-tape, with a separate party running the levels as described. However, in open country all three preliminary survey operations (running line, running levels, and locating details) may be done at the same time, by a single party using transit-stadia. The party, set up on the traverse, first measures the horizontal angles, vertical angles, and stadia distances applying to the traverse ahead; then takes side shots applying to the cross-section elevations and details.

As the preliminary survey proceeds, each day's work is plotted on a preliminary PROFILE and a preliminary MAP. The profile shows the plotted elevations of existing ground along the traverse line; the map shows the topography and other detail along the line, including contours. A commonly used scale for highway preliminary profiles is horizontal 1 in. = 100 ft, vertical 1 in. = 10 ft. The contour interval for a highway preliminary map varies according to the slope or irregularity of the ground; the average for ordinary country is 5 ft, but in level country it may be 2 ft or even 1 ft, and in rough country may be 10 ft or more.

Final Location Survey

From the preliminary survey data, one of the alternative routes suggested by reconnaissance is selected and the others are eliminated. Along the selected route, a tentative location for the highway centerline (including curves) is chosen. This location is still tentative, because circumstances which develop in the course of the final location survey may require departures. Usually the tentative horizontal location is drawn in on the preliminary map, and the tentative vertical location on the preliminary profile. In this case these are familiarly called the PAPER locations.

A number of considerations influence the selection of the horizontal location and grade of a highway. Some of the most important are:

1. Keeping changes in direction at a minimum, and making unavoidable changes in direction as gradual as possible.
2. Keeping changes in elevation at a minimum, and making unavoidable changes in elevation as gradual as possible.
3. Making total volumes of cut and fill as small as possible.
4. Minimizing haul expense by making the distance from borrow pit to cut in each case as small as possible and using as little borrow as possible. An attempt is made to set line and grade in a manner which will best facilitate the filling of hollows with fill taken from nearby high points along the traverse.
5. Providing for adequate drainage slopes.

The first task of the final location survey field party might be described as the task of adjusting the preliminary traverse to fit the requirements of the paper location traverse. The paper location traverse may, for parts of its length, coincide with the preliminary traverse; for these sections, no adjustment is necessary. For those sections along which the two do not coincide, the field party has the problem of locating, on the ground, the paper location traverse so that its ground location will bear the same relation to that of the preliminary traverse that the two bear to each other on the preliminary map.

This relation may be determined by any of the methods of tying in points described in *Engineering Aid 3 & 2*. Consider figure 6-1, for example. In this figure the line through stations C, D, and E represents the preliminary traverse. The paper location traverse coincides with the preliminary traverse to station C', but then runs S 73°30' E, 580.36 ft, to station D', and thence to rejoin the preliminary traverse at station E.

One way of locating station D' on the ground would be by perpendicular offset from the preliminary traverse, as indicated by the dotted line SD'. A right triangle solution will locate S on the preliminary traverse, and determine the length of SD'. The bearings of C'S and C'D indicate that angle A must measure 29°42'. The length of C'S amounts to 580.36 cos 29°42', or 580.36(0.868632), or 504.12 ft. Therefore, point S will be located 504.12 ft from station C', or at station 18 + 29.16.

The length of SD' amounts to 580.36 sin 29°42', or 580.36(0.495459), or 287.54 ft. Therefore, to locate station D' you would set up a transit at station 18 + 29.16 on the preliminary traverse, turn 90°, and lay off 287.54 ft from S.

Another way to locate station D' would be by triangular intersection from stations C' and E. The bearings indicate that the size of angle B is 26°54'. Angle A measures 29°42'. Set one transit up at C', backsight on D, and turn 29°42' to the right. Set up another transit at E, backsight on D, and turn 26°54' to the left. A range pole sighted through both telescopes will be located on station D'. Measure the distance C'D' to check if it measures 580.36 ft—the paper distance.

Stakes are usually set at all full stations, and at all P.I.'s, P.C.'s, and P.T.'s. In general, stakes are set at 50-ft stations on horizontal curves. Where the location traverse diverges from the preliminary traverse, profile levels and cross-sections for this portion are taken. Then, a corrected location plan is prepared; in most cases this correction is just integrated to the existing plan. As a result of a study of these departures from the preliminary traverse, further adjustments in line or grade may be ordered. When these have been made in the field, and on the location plan and profile, the final location survey is completed.

OTHER ROUTE SURVEYS

Other structures which follow routes are utilities lines, which may be broadly divided into OVERHEAD power and communications lines and UNDERGROUND power/communications lines; sewerlines; and water, gas, and fuel lines. The character of the route survey for a utility will vary, of course, with the circumstances. A sanitary sewer, for example, will follow the streets on which the buildings it serves are

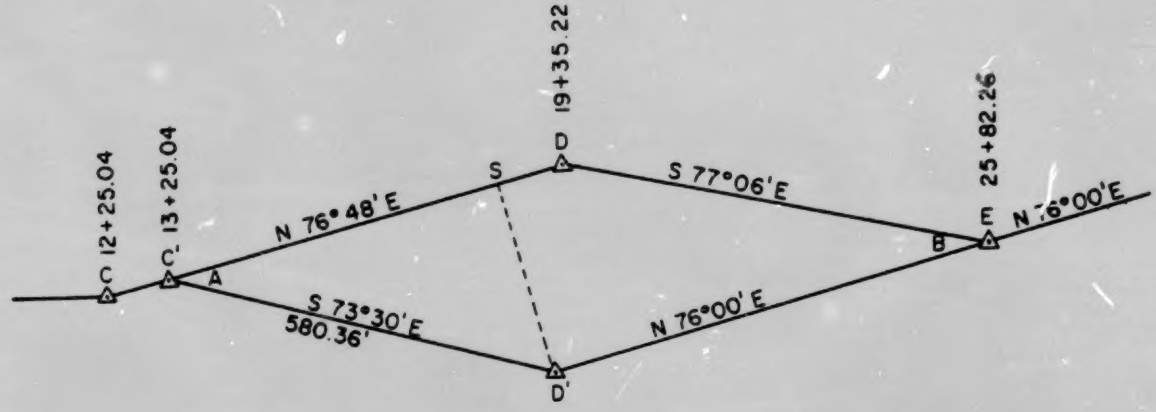

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Figure 6-1.—Tying final location traverse to preliminary traverse.

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located; consequently, reconnaissance and preliminary surveys are seldom necessary for one of these. The same applies, in general, to the DISTRIBUTION part of a power, gas, water, or fuel line. The location of one of these is controlled by the locations of the buildings it serves.

If utilities lines already exist in an area, they are shown on UTILITIES MAPS. A separate map is generally used to show the principal features of each utility. Small-scale maps show locations, materials, pipe sizes, and other information relating to the main transmission, collection, and distribution systems. Minor construction details and service connections are shown on larger-scale detail plans.

A utilities project often involves extending an existing line—as, for example, tying in a line from new housing to an existing sanitary sewer. The first requirement in a case of this kind is a study of the existing utilities maps.

ABOVEGROUND UTILITIES ROUTE SURVEYS

Aboveground utilities are usually electrical power or communications lines, strung on poles or towers. The location of the DISTRIBUTION (service) part of one of these is usually controlled by the locations of the buildings it serves—meaning that the service or distribution line will usually follow the streets on which the buildings are located. For the TRANSMISSION part, however, judgment in the selection of a route is usually required. By transmission part we mean that part of a line which carries (for example) high-voltage electrical power from the power plant to points from which the power is distributed to consumer outlets.

The route survey for a power transmission line, then, may be divided, like that for a highway, into reconnaissance, preliminary, and final location surveys. Controlling considerations are, of course, different from those for a highway. For a tower line, construction economy requires that changes in direction be kept at a minimum, for the reason that a tower located where a line changes direction must stand a higher stress than one located in a straight-line part of the line. In general, tower construction in level country is cheaper than construction in broken country; however, the line may be run over broken country to minimize changes in direction, or to make it shorter, or to follow a line where the cost of obtaining right-of-way is cheap. To facilitate access for construction and future maintenance, it is desirable that lines be located adjacent to existing roads.

When running the preliminary survey, incorporate all pertinent topographic information into the field notes. Note particularly any existing overhead or subsurface lines and indicate whether they are power or communications lines. Locate such features as marshes, streams, forests, roads, railways, power plants, laboratories, and adjacent military camps or bases.

Pole Line Surveys

When the route has been selected on the basis of the reconnaissance data, a plan and profile are plotted. The plan shows the route the line will follow and the significant topography adjacent thereto. The profile shows the ground elevation along the line and the top elevations of the poles. These are generally set in accordance with a specified minimum allowable clearance between the ground line and the top of a pole. In open country the minimum clearance for a line crossing roads, railroads, or waterways is usually 14 ft; in built-up country it is usually 18 ft. Minimum allowable clearance between two crossing powerlines is usually 4 ft; for two communications lines it is usually 2 ft apart.

When a power or communication line crosses a highway or railroad, poles adjacent to the highway or road are usually required to be set a minimum of 12 ft back from the highway shoulders or railroad rails.

Poles are numbered consecutively in accordance with a specified theoretical span between adjacent poles—regardless of whether or not there is actually a pole set at each of these intervals. Theoretically, the spacing is usually 150 ft. The line might cross a stream wide enough to cause the span between adjacent poles to be—say—400 ft. Suppose the pole number on one side of the stream was 65. The next adjacent pole on the other side would be, not #66, but #68.

Each pole location is marked with a hub on the line; the hub may be offset. On the guard stake you put the pole number, the line elevation, and the distance from the top of the hub to the top of the pole, obtained from the profile. This is usually simply designated as a plus, as: +25 ft.

Tower Line Surveys

High voltage lines are often supported by TOWERS, a tower being a broad-based steel structure 35 ft or more in height. When a change in direction in a tower line is unavoidable, it is made gradually, in as small angular increments as possible. Suppose, for example, a change in direction of 90° was required. Instead of an abrupt change in direction of 90°, towers would be set so as to cause the line to follow a gradual curve in a succession of chords around an arc of 90°.

Each tower in a curve of this kind must be placed at an angle which will balance the lateral pull of the cables in and out of the tower. This is done by locating the centerline of the tower so as to bisect the angle between the lines as shown in figure 6-2.

BELOWGROUND UTILITIES ROUTE SURVEYS

When man-made structures are erected in a certain area, it is necessary to plan, design, and construct an adequate drainage system. Generally, an underground drainage system is the most desirable means in removing surface water effectively from operating areas. An open drainage system, like a ditch, is economical; however, when not properly maintained, it is unsightly and unsafe. Sometimes an open drainage system also causes erosion, thus resulting in failures to nearby structures. Flooding caused by an inadequate drainage system is the most prevalent cause leading to the rapid deterioration of roads and airfields. The construction and installation of drainage structures will be discussed later in this chapter. At this point we are mainly interested in drainage systems and types of drainage.

Drainage System

SANITARY sewers carry waste from buildings to points of disposal; STORM sewers carry surface runoff water to natural water courses or basins. In either case the utility line must have a GRADIENT–that is, a downward slope toward the disposal point, just sufficiently steep enough to ensure a gravity flow of waste and water through the pipes. This gradient is supplied by the designing engineer.

Natural Drainage

To understand the controlling considerations affecting the location and other design features of a storm sewer, you must know something about what might be called the mechanics of water drainage from the earth's surface.

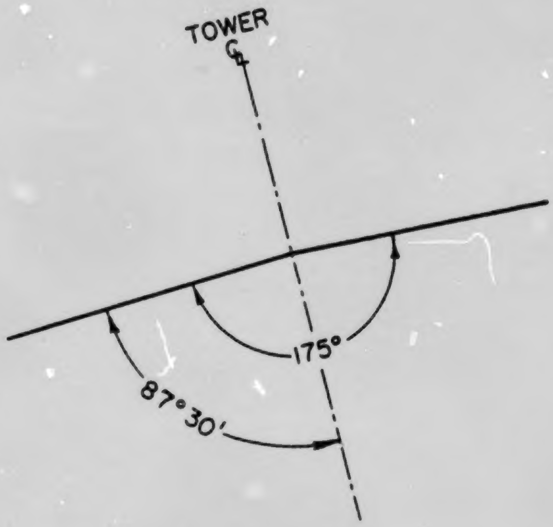

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Figure 6-2.–Balancing the stresses on a tower.

When rainwater falls on the earth's surface, some of it is absorbed in the ground. The amount thus absorbed will vary, of course, according to the physical characteristics of the surface. In sandy soil, for instance, a large amount will be absorbed; on a concrete surface, on the other hand, absorption will be negligible.

Of the water which is not absorbed in the ground some will evaporate, and some, absorbed through the roots and exuded onto the leaves of plants, will additionally dissipate through a process called TRANSPIRATION.

The water which remains after absorption, evaporation, and transpiration is technically known as RUNOFF. This term relates to the fact that this water, under the influence of gravity, will make its way (i.e. "run-off") through natural channels to the lowest point it can attain. To put this in terms of a general scientific principle, water will, whenever it can, seek its own level. The general, final level which unimpeded water on the earth's surface will seek is sea level, and the rivers of the earth, most of which empty into the sea, are the earth's principal drainage channels. However, not all of the earth's runoff reaches the great oceans; some of it is caught in land-locked lakes, ponds, and other nonflowing inland bodies of water.

Let's consider, now, a point high in the mountains somewhere. As rain falls in the area around this point, the runoff runs down the slopes of a small gulley, and forms a small stream which finds a channel downward through the ravine between two ridges. As the stream proceeds on its course, it picks up more and more water draining in similar fashion from high points in the area through which the stream is passing. As a result of this continuing accumulation of runoff, the stream becomes larger, until eventually it either becomes or joins a large river making its way to the sea–or it may finally empty into a lake or other inland body of water.

The natural channels through which this runoff passes will generally contain and dispose of all the runoff in normal weather circumstances. However, it is commonly the case that during the winter in the high mountains runoff is interrupted by snow conditions–that is, instead of running off, the potential runoff accumulates in the shape of snow. When this accumulated mass melts in the spring, the runoff often attains proportions which overwhelm the natural channels, causing flooding of surrounding areas. In the same fashion, unusually heavy rainfall may overtax the natural channels.

Artificial Drainage

When artificial structures are introduced into an area, the natural drainage arrangements of the area are upset. When, for example, an area originally containing many hills and ridges is graded off flat, the previously existing natural drainage channels are removed and much of the effect of gravity on runoff is lost. When an area of natural soil is covered by artificial paving, much water which was previously absorbed will now present drainage problems.

In short, when man-made structures such as bridges, buildings, etc. are erected in an area, it is usually necessary to design and construct an artificial drainage system to offset the extent to which the natural drainage system has been upset. Storm sewers are usually the primary feature of an artificial drainage system; however, there are other features, such as drainage DITCHES. Both storm sewers and ditches carry surface run off. The only real difference between a drainage ditch and a storm sewer is the fact that the ditch lies on the surface and the storm sewer below the surface.

Similarly it might be said that there is no essential difference in mechanical principle between an artificial and a natural drainage system. Like a natural channel, an artificial channel must slope downward, and must become progressively larger as it proceeds along its course, picking up more runoff as it goes. Like a natural system, an artificial system must reach a disposal point–usually a stream whose ultimate destination is the sea or a standing inland body of water. At the terminal point of the system where the accumulated runoff discharges into the disposal point, the runoff itself is technically known as DISCHARGE. The discharge point in the system is called the OUTFALL.

Parts of an Artificial Drainage System

A surface drainage system consists principally of DITCHES, which form the drainage channels. A ditch may consist simply of a depression formed in the natural soil, or it may be a PAVED ditch. Where a ditch must pass under a structure (such as a highway embankment, for example), an opening called a CULVERT is constructed. A PIPE culvert has a circular opening; a BOX culvert has a rectangular opening. Walls constructed at the ends of a culvert are called END walls. An end wall, running perpendicular to the line through the culvert, may have extensions called WINGS (or WING WALLS) running at an oblique angle to the line through the culvert.

An underground drainage system (that is, a storm sewer) consists, broadly speaking, of a buried pipeline called the TRUNK or MAIN, and a series of STORM WATER INLETS which admit surface runoff into the pipeline. An inlet consists of a surface opening which admits the surface water runoff, and an inner chamber called a BOX (sometimes a CATCH BASIN). A box is usually rectangular, but may be cylindrical. An inlet with surface opening in the side of a curb is called a CURB inlet; a working drawing of a curb inlet is shown in figure 6-3. An inlet with a horizontal surface opening covered by a grating is called a GRATE (sometimes a DROP) inlet. A general term applied in some areas to an inlet which is neither a curb nor a grate inlet is YARD inlet.

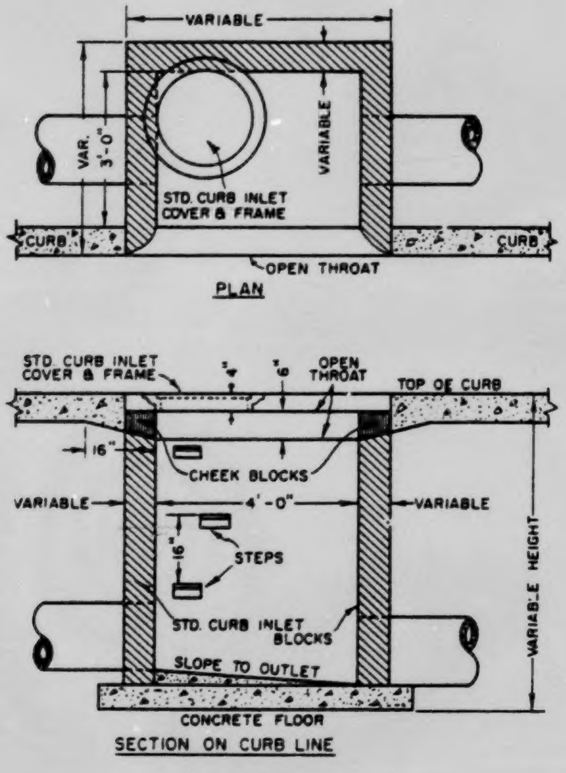

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Figure 6-3.—Working drawing for typical curb inlet.

Technically speaking, the term "storm sewer" applies to the pipeline; the inlets are called APPURTENANCES. There are other appurtenances, the most common of which are MANHOLES and JUNCTION BOXES. A manhole is a box which is installed, of necessity, at a point where the trunk changes direction, gradient, or both. The term "manhole" originally related to the access opening at one of these points; however, a curb inlet and a junction box nearly always have a similar access opening, for cleaning, inspection, and maintenance purposes. One of these openings is often called a "manhole," regardless of where it is located. However, strictly speaking, the access opening on a curb inlet should be called a curb inlet opening; that on a junction box a junction box opening. Distances between manholes are normally 300 ft, but this distance may be extended to a maximum 500 ft if the specification requires it.

The structure at an access opening consists of the manhole (or curb inlet, or junction box) COVER and a supporting metal casting called the FRAME. A frame for a circular cover is shown in figure 6-4. Some covers are rectangular. The frame usually rests on one or more courses of ADJUSTING blocks, so that the rim elevation of the cover can be varied slightly to fit the surface grade elevation by varying the vertical dimensions, or the number of courses, of the adjusting blocks.

A junction box is similar to a manhole, but is installed of necessity at a point where two or more trunk lines converge. The walls of an inlet, manhole, or junction box may be constructed of special concrete masonry units or of cast-in-place concrete. The bottom consists of a formed slab, sloped in the direction of the line gradient, and often shaped with channels for carrying the water across the box from the inflowing pipe to the outflowing pipe.

Drainage Design

A complete description of the many problems involved in drainage design cannot be given in this course. We can only discuss enough of the

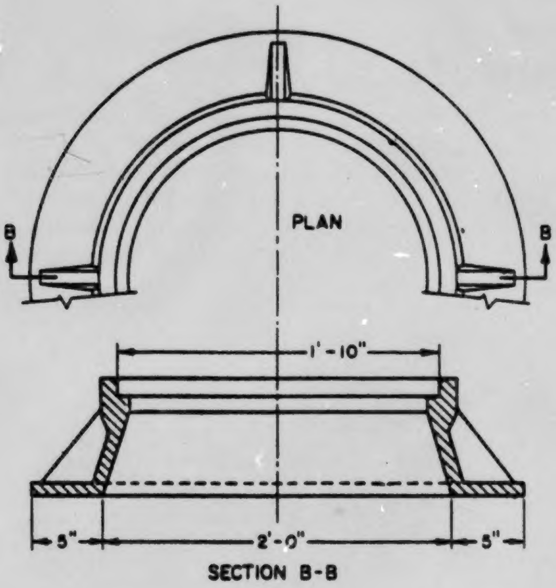

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Figure 6-4.—Frame for an access opening.

highlights to make it possible for you, as field party chief, to assist the design officer intelligently. In drainage design, the topography of the whole area concerned is closely studied. If satisfactory topographic maps do not exist, such must be made. Obviously, of course, both the existing terrain and the terrain as it will exist after construction are considered.

The first consideration with regard to the drainage problem at a particular point is the maximum quantity of runoff which may be expected at that point. This maximum quantity is familiarly called the "Q". The point in question is called the POINT OF CONCENTRATION. A study of the map (particularly of the relief) determines the limits of the CONTRIBUTING AREA for that point—that is, the boundaries of the area around the point from which water will drain toward the point of concentration. Next, the runoff for this area is determined. The quantity of this runoff will depend upon a number of factors, one of which is the INTENSITY of the rainfall in the contributing area UNDER DESIGN CONDITIONS.

The term "intensity" signifies a RATE of rainfall: a 10-minute cloudburst which dumps 1 inch of water on an area means a rate of 6 inches per hour, there being six 10-minute intervals in an hour. The numerical value of intensity for a given contributing area is affected by (1) the duration of the storm, and (2) the design frequency of the storm. The duration of the storm is presumed to be equal to the length of time it takes a drop of water at the outermost limit of the contributing area to reach the point of concentration; it is assumed that at this time the entire area is contributing runoff to the point of concentration. Design frequency refers to how bad a storm the system can carry without flooding—for example, a storm that can be expected only once in 2 years, or in 5 years, or in 10 years. It is seldom economical to design for the worst possible storm; the design frequency selected will vary according to the extent to which occasional flooding of the system will cause damage. Obviously, occasional flooding of the ground floor of a shop containing valuable articles would be more serious than occasional flooding of, say, a tennis court.

The system, then, may be designed for the type of storm which occurs only every 2 years, or every 10 years—this is familiarly called "designing for a 2-year storm" or "for a 10-year storm." If weather records exist, design intensity can be based on these. If they don't, estimates must be based on the experience of inhabitants, on judgment, or on both.

Now, the product of the design intensity times the area (in acres) of the contributing area gives the amount of water which would accumulate as runoff at the point of concentration if all this water ran off. However, some of the water will be absorbed, some will evaporate, and some will dissipate through transpiration. Therefore, the product of the contributing area (A), times the design intensity (i), is reduced by multiplying by a factor called the RUNOFF COEFFICIENT (C). The result is the amount of the product of A times i which will remain as runoff after absorption, evaporation, and transpiration. In general, to get the runoff coefficient you refer to tables similar to the one shown table 6-1. Such tables are based upon studies made in the past with regard to the perviousness or imperviousness of various types of surfaces, and apply the table values to estimates of the proportion of pervious to impervious areas in the contributing area.

Table 6-1.–Surface Runoff Factors

Types of surface	*Coefficients*
Pavements (concrete or asphalt)	0.70 to 0.95.
Gravel or macadam pavements	0.35 to 0.70.
Impervious soils[1]	0.40 to 0.65.
Impervious soils, with turf[1]	0.30 to 0.55.
Slightly pervious soils[1]	0.15 to 0.40.
Pervious soils[1]	0.01 to 0.10.
Wooded areas depending on surface slope and soil cover	0.01 to 0.20.

[1] For slopes from 1 to 2 percent.

Note. The figures given are for comparatively level ground. For slopes greater than 1 in 50 (2%) the coefficient should be increased by 0.2 for every 2 percent of slope, but coefficient cannot exceed 1.0.

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In sum, then, the Q (quantity of runoff, in cu ft per second, accumulating at the point of concentration during the design storm) is determined from the formula ACi, A being the area of the contributing area in acres, C being the runoff coefficient expressed as a percentage (as, 0.40), and i being the intensity of rainfall, in inches per hour, in the contributing area during the design storm.

You may note that in this formula acres is multiplied times inches per hour and the answer is reported in cubic feet per second—an apparent inconsistency in units. The use of these units is possible because the flow of 1 inch of water from 1 acre in 1 hour is almost equal in numerical value to the flow of 1 cubic foot of water in 1 second. This fact may be demonstrated by assuming an area of 1 acre, a coefficient of runoff of 1, and a rainfall intensity of 1 inch per hour.

$$Q = ACi = 1 \text{ acre} \times 1 \times \frac{1 \text{ inch}}{1 \text{ hour}}$$

$$Q = \frac{43560 \text{ sq ft} \times 1/12 \text{ foot}}{3600 \text{ seconds}} = \frac{3630 \text{ cu ft}}{3600 \text{ seconds}}$$

$$Q = 1.008 \text{ cubic feet per second}$$

Generally, the first point of concentration is the highest point on or near the structure where runoff will accumulate, under design conditions, to an extent requiring artificial drainage. Therefore, the system will begin at this point. The next point of concentration will be the next adjacent point where runoff thus accumulates. Between the first and second points the system will be designed to carry the Q which accumulates at the first point. Between the second and the third point, it must be designed to carry the Q which accumulates at the first two points. Thus, as the system proceeds along its course, the Q it must be designed to carry increases, which means that the carrying capacity of the system must progressively increase as well. The nature of the structure may require branching trunks, converging at various points; in a case of this kind, of course, the pipe beyond a junction box must be designed to carry the Q brought into the junction box by the contributing trunks above it.

The maximum quantity of water which will flow at a given rate through a drain pipe is controlled by (1) the size of the pipe, (2) the gradient (slope) of the pipe, and (3) the classification or type of the pipe. Item 3 is principally a matter of friction; everything else being equal, water will flow more rapidly through smooth-walled concrete pipe (for example) than it will through corrugated metal pipe. If the system contains the same type of pipe throughout, the roughness coefficient will remain the same for all sections. This coefficient can be found in tables for pipe made of all the commonly used materials.

Drain water will flow faster through a steeply sloped pipe than it will through a pipe that is nearly horizontal. Therefore, when Q increases to the point where the flow capacity of the system must be increased, this may be accomplished by increasing the slope. The extent to which this can be done, however, depends on the circumstances. If the ground is level and the line is long, a large slope percentage will soon carry the pipe too far down into the ground. But if the ground itself slopes downward, the pipe can be carried indefinitely at the same downslope.

The flow capacity of pipe increases with the inside diameter of the pipe. Therefore, the flow capacity may also be increased by using larger pipe. If the alternative exists of increasing either the slope or the diameter of the pipe, the alternative selected will be, of course, the one which is cheaper in view of all the long-run circumstances.

All of these considerations influence the selection of pipe sizes, gradients, and elevations. The route followed by the trunk line is also affected. Normally this route would follow, more or less, the course from one point of concentration to the next. However, it is often necessary or desirable to diverge from this route, to avoid structures, to take advantage of sloping ground, to avoid areas of difficult excavation, or for some other reason. When this is the case, runoff from inlets not located over the trunk is carried to the trunk by LATERAL or BRANCH pipelines. A common use of laterals or branches is to carry runoff from an inlet on one side of a highway to a trunk located along the opposite side.

Design Computations

The flow capacity, in cu ft per second, of a given section of pipeline is determined from the formula aV, a being the section area in sq ft of the flowing stream and V the velocity of speed of flow in fps. If it is assumed that the pipe will flow full, a is the section area inside the pipe wall. For a 24-in. pipe a is π times R^2, or π times 1^2, or 3.14 sq. ft.

To determine V you use another formula, as follows:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

The symbol n in this formula is the roughness coefficient, usually obtained from tables based on the character and size of the pipe. R (which stands for HYDRAULIC RADIUS) is determined by dividing a by the length of the WETTED PERIMETER. The wetted perimeter is the portion of the inner circumference of the pipe which is under water when the stream is flowing at design maximum. If the pipe is presumed to flow full, the wetted perimeter is equal to the circumference. Therefore, for a 24-in. pipe, flowing full, the wetted perimeter is 2 ft $\times$ π, or 6.28 ft. R is $\frac{a}{6.28}$ or $\frac{3.14 \text{ sq ft.}}{6.28 \text{ ft}}$, or 0.50 ft. R to the 2/3 power means the cube root of R^2, or the cube root of 0.25, or 0.63.

S in the formula means the gradient of the pipe IN FEET PER FOOT. A 2 percent gradient means a drop of 2 ft in 100 ft, which in turn means a drop of 0.02 ft in 1 ft. S to the 1/2 power means the square root of S, which in this case is the square root of 0.02, or 0.14.

Assume that the roughness coefficient (n) is 0.013. Substituting the known values in the formula for V (velocity), we have:

$$V = \frac{1.486}{0.013} \times 0.63 \times 0.14$$

If you work this out, you will find that V comes to 10.01 fps. The flow capacity, then, is a x V, or 3.14 sq ft x 10.01 fps, or 31.43 cfps.

This is the flow capacity of a section in which the gradient and the size of the pipe are given. The design problem, however, is more likely to involve determining the gradient and size of the pipe for handling a predetermined quantity of runoff. One way of doing this is by first determining the minimum slopes required for pipe through a range of sizes, as follows.

Figure 6-5 shows a flow diagram of a storm sewerline running along a highway. The trunk begins at inlet 2 and runs through inlets 3 and 6 to the outfall. Inlets 4 and 5 admit runoff to laterals connected to the boxes at 3 and 6.

The dotted lines show the limits of the contributing areas for each of the inlets. Suppose, now, that the Q for each area, under design conditions, is as follows:

Inlet No.	Q (cfs)
1	4.5
2	3.6
3	3.0
4	2.7
5	3.0
6	3.0

The total Q at the outfall is the sum of these, or 19.8 cfs. The next problem is to determine the Q which must be handled by each of the separate sections (called LINES) in the system. The line from 1-2 will handle only the Q from contributing area 1, or 4.5 cfs. The line from 2-3, however, will carry the sum of the Q's from areas 1 and 2, or 4.5 plus 3.6, or 8.1 cfs. The line from 4-3 will carry only the Q from area 4,

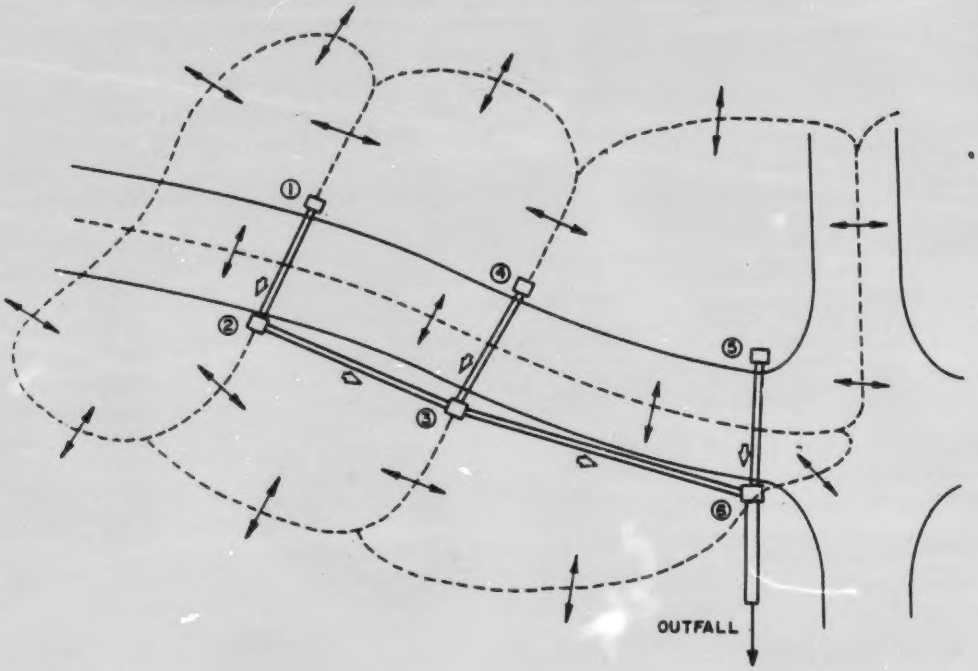

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Figure 6-5.—Flow diagram inlet of a storm sewerline system.

or 2.7 cfs. But the line 3-6 will carry the Q's from areas 1, 2, 3, and 4, or 4.5 + 3.6 + 3.0 + 2.7, or 13.8 cfs. The line 5-6 will carry only the Q from area 5, or 3.0 cfs.

If you add these together, you will find that the sum is again 19.8 cfs—equal, as it should be, to the previously determined total Q at the outfall.

The next step is to determine, for each line, the MINIMUM slope required for pipe ranging through all the sizes considered feasible. Drain pipe is made in sizes from 4 in. to about 108 in. in diameter. As a rule the use of pipe smaller than a certain minimum size (usually about 12- or 15-in.) is not permitted for storm sewers, because of the possibility of clogging with debris. Also, for pipe of a given size and character, there is usually a maximum permissible velocity of flow, beyond which the pipe would be seriously eroded by the velocity of the flowing stream. This means that for a pipe of a given size and character, the maximum feasible slope is the slope which permits the maximum permissible velocity of flow.

The largest pipe to be considered would be the largest required to carry the total Q at the outfall at the minimum permissible slope. In working these design calculations, engineers use various types of circular slide-rule or graphic calculators called NOMOGRAPHS, from which formula solutions may be obtained by inspection. If you enter one of these with the roughness coefficient, the size of the pipe, and the Q for the line, you can determine the minimum slope for that Q, size of pipe, and roughness coefficient by inspection.

Assuming now a roughness coefficient of 0.013 and a total Q at the outfall of 19.8 cfs, then a nomograph calculation indicates that a 27-in. pipe will carry this Q at a slope of 0.41 percent. Assuming the minimum specified slope to be 0.5 percent, then 27-in. will be adequate for this minimum specified slope, and is the largest pipe that needs to be considered. Assume that the smallest permissible pipe is 15-in. Pipe in ranges from 15-in. to about 30-in. is usually available in sizes 3-in. apart—that is, 15-in., 18-in., 21-in., and so on. Beyond 30-in. the sizes usually increase in 6-in. increments.

We are going to determine the minimum slopes required for pipe from 15-in. to 27-in. for each of the lines in the system. This data is entered in a form as follows:

		Minimum slope (%) for Following Sizes				
Line	Q (cfs)	15"	18"	21"	24"	27"
1-2	4.5	0.5	–	–	–	–
2-3	8.1	1.6	0.6	0.26	–	–
4-3	2.7	0.17	–	–	–	–
3-6	13.8	4.6	1.7	0.76	0.38	–
5-6	3.0	0.22	–	–	–	–
6 Out	19.8	9.3	3.5	1.6	0.77	0.41

This tabulation indicates that, for the Q for line 1-2, a 15-in. pipe at the flattest permissible slope is sufficiently large. Therefore, there is no point in calculating the minimum slope for larger pipe for this line.

For line 2-3, on the other hand, a 15-in. pipe would have to be placed at a slope of at least 1.6 percent. If the character of the ground were such as to make this too steep a slope (taking the pipe too far down in the ground), it might be more economical to use 18-in. pipe, which would carry the Q at a slope of only 0.6 percent. In deciding between these alternatives, the excavating cost would be weighed against the increase in price for larger pipe. There are similar alternatives for line 3-6 and for the outfall line.

In general, a design engineer determines by studying the profile, what the maximum possible economically feasible slope is for each line in the system. For a long line running in relatively level ground, the slope is obviously more limited that it is for a line running in ground which slopes in the direction of the line. However, even when the ground slopes radically downward, there is a limit beyond which the pipe may not be economically sloped. A slope of more than this causes the water to attain a velocity which rapidly erodes concrete pipe. When the engineer has determined the maximum slope which is economically feasible for each line, he selects from the table the smallest pipe capable of handling the Q for that line at that slope.

For the determination of required cross-sectional areas and capacities of culverts for most Theater of Operations roads and outlying areas of an airfield, where accurate computation of runoff is impracticable. Talbot's formula can be used. This formula is an approximate method for computing the cross-sectional area of the proposed pipe or culvert. It is stated as:

$$A = C\sqrt[4]{D^3}$$

where: A = area of waterway opening in square feet
C = a coefficient that depends upon the slope, shape, and character of area to be drained
D = drainage area in acres.

This formula is recommended only for small structures requiring a waterway opening not greater than 400 sq ft. In addition, the formula is intended only for a rainfall intensity of 4 inches per hour for 1 hour or less. For locations having greater intensities than this, the required opening may be computed by dividing the area of the drainage structure obtained from the formula by 4; then the result is multiplied by the intensity of rainfall to be expected at the given location. The key to the use of the formula is the judgment exercised by the engineer in the choice of a value for the coefficient "C". Normal values for C are as follows:

C = 0.2 flat areas not affected by cumulated snow, and where the length of the valley drained is several times the width.
C = 0.35 for gently rolling farmland, where the length of the valley is about 3 or 4 times the width.
C = 0.7 for rough hilly areas having moderate slopes.
C = 1.0 for steep barren areas having abrupt slopes, and for moderately mountainous areas.

The value of the coefficient C is influenced by the shape of the drainage area, the side slopes, the length of the valleys, and the character and culture of the ground. All of these factors affect the proportion of runoff and its time of concentration at the culvert. Therefore, the engineer must adjust the value of C to suit each case. The value of C should be increased as the lengths of the valleys decrease in proportion to their widths, and vice versa. As side slopes steepen, C also should increase. Heavy shrub growth would decrease the value of C over cultivated farmland, whereas rocky or barren slopes would increase the value of C. Predominately sandy or gravelly soils tend to decrease C, whereas heavy clay soils tend to increase C. A value of 1.0 is satisfactory for moderately mountainous terrain, or for reasonably steep barren areas with abrupt slopes up to 10 percent. The formula should not be used for precipitous rocky mountain areas where C would be greater than 1.0. The drainage areas may be obtained from the map of the area involved by planimeter, or by dividing the area into several triangles and/or rectangles.

Transition Loss

As shown in figure 6-6, the line connecting the lowest inside points on a pipe in place is called the INVERT; that connecting the highest inside points is called the CROWN. The SPRING LINE lies halfway between the invert and the crown.

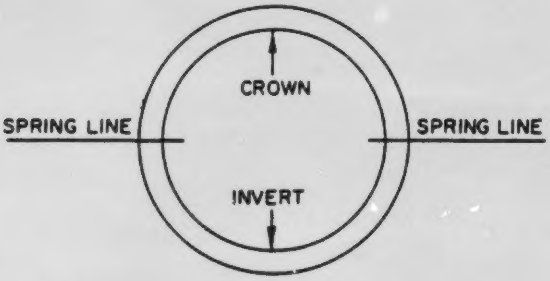

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Figure 6-6.—Cross-section view of a storm sewer pipe.

A point where a sewerline changes pipe size, direction, or gradient is called a TRANSITION. A point where two or more lines converge is also a transition. Now, at any transition point there is a loss of energy in the stream flow. Turbulence in the box caused by a change in direction (either horizontal or vertical) and by the box structure itself cuts down the rate of flow, to the extent that the outflowing pipe will not flow to design capacity.

This type of transition loss is estimated, as well as it can be, and is usually taken into account in the design of the outflowing line. Another type of transition loss is easier to determine. Figure 6-7 illustrates this type. This figure shows a section through a manhole. Notice first that the pipe is presumed to run to the manhole centerline; it is actually, of course, cut off in line with the inner wall of the box.

The specified elevation for a pipe is the elevation of the invert, and the elevation at a box means the invert elevation AT THE CENTERLINE OF THE BOX. Now, at the manhole shown in figure 6-7, the inflowing and outflowing pipes are the same size (18-in.). However, because of an increase in slope in the outflowing pipe, that pipe, flowing full, allows the stream to flow at a higher velocity. You can see that V for the inflowing pipe is 5 fps, while V for the outflowing pipe is 8 fps.

If the stream, when it reaches the outflowing pipe, is still flowing at only 5 fps, the outflowing pipe will not flow full. Therefore, the box is designed to step up the flow to the velocity of which the outflowing pipe is capable. This is accomplished by DROPPING THE INVERTS—that is, by placing the invert out at a lower elevation than that of the invert in.

The amount of the drop can be calculated, by determining the difference in the respective VELOCITY HEADS of the two pipes. Velocity head can be determined from the formula $\frac{V^2}{2g}$. V in this formula is the velocity of which the pipe, flowing full, is capable; this is determined by formula as previously explained, or from a nomograph. The symbol g in the formula means the force of gravity, which is 32 ft/sec^2.

For the pipes shown in figure 6-7 then, the velocity heads are as follows:

$$\text{Inflowing pipe} = \frac{25}{64} = 0.39 \text{ ft.}$$

$$\text{Outflowing pipe} = \frac{64}{64} = 1.00 \text{ ft.}$$

The amount of the drop is the difference between these two, or 0.61 ft. The bottom of the box would, of course, be filled with cast concrete or rubble pargetted with mortar, to channel the stream smoothly from the inflowing pipe to the outflowing pipe; for the sake of clearness, this material is omitted in figure 6-7.

The computation just described gives you the amount of invert drop WHEN THE PIPES ARE OF THE SAME SIZE. When the outflowing pipe is LARGER than the inflowing pipe, you ADD the difference in diameters of the pipe to what the drop would be for pipe of the same size. Suppose, for example, that in figure 6-7 the outflowing pipe was 24-in. instead of 18-in. The difference in diameters here is 6 in., or 0.50 ft, and the invert drop would be 0.61 ft plus 0.50 ft, or 1.11 ft.

When the outflowing pipe is SMALLER than the inflowing pipe, you SUBTRACT the difference in diameters of the pipe from what the drop would be for pipe of the same size. Suppose that in figure 6-7 the outflowing pipe was 15-in. instead of 18-in. The difference in diameters is 3 in., or 0.25 ft, and the invert drop would be 0.61 minus 0.25, or 0.36 ft.

It may occasionally happen that the difference in diameters is larger than what the drop for pipe of the same size would be, thus

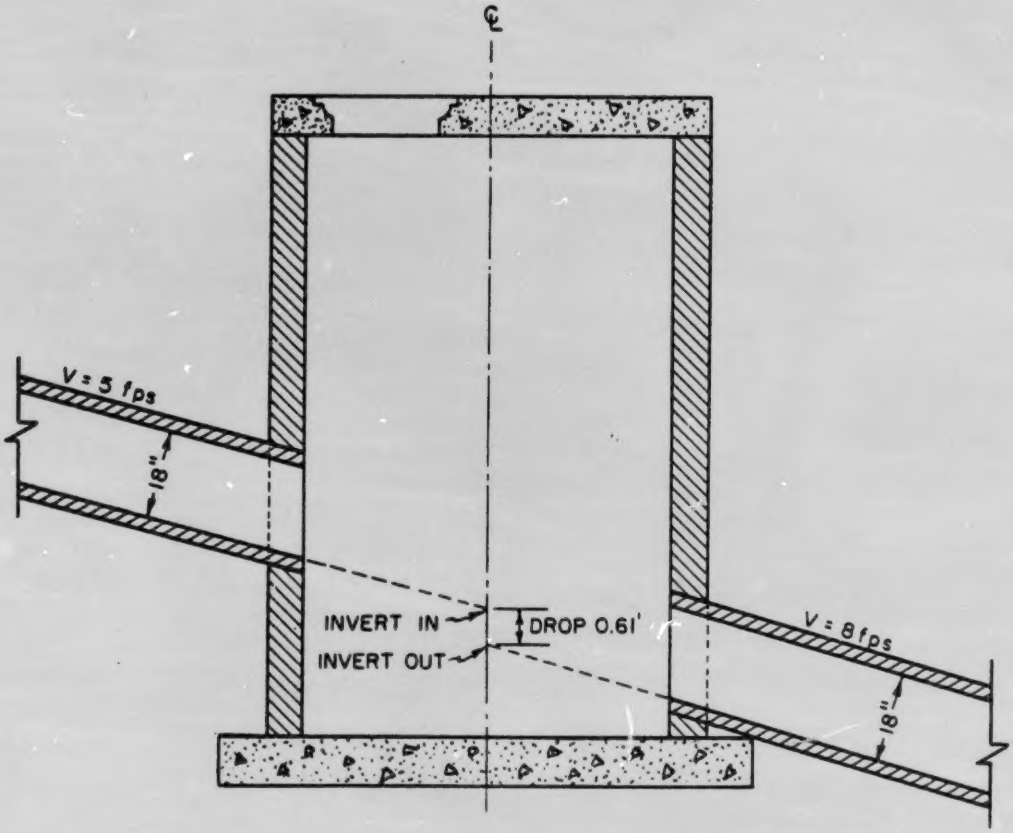

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Figure 6-7.—Dropping inverts.

requiring, by computational theory, a rise rather than a drop for the outflowing pipe invert. In a case of this kind (which is rare), you follow a general rule to the effect that an invert out is never placed at a higher elevation than an invert in.

Storm Sewer Route Survey

The character of the route survey for a storm sewer depends on the circumstances. The nature of the ground may be such as to indicate, without the necessity for reconnaissance and preliminary location surveys, just where the line must go. This is likely to be the case in a development area—that is, an area which will be closely built up, and in which the lines of the streets and locations of the buildings have already been determined. In these circumstances, the reconnaissance and preliminary surveys might be said to be done on paper.

It is often the case, on the other hand, that a line—or parts of it—must be run for considerable distances over rough, irregular country. In these circumstances the route survey, consists of reconnaissance, preliminary location, and final location surveys. If topographic maps of the area exist, they are studied to determine the general area along which the line will be run. If no such maps exist, a reconnaissance party must select one or more feasible route areas, run random traverses through these, and collect enough topo data to make the planning of a tentative route possible.

After this data has been studied, a tentative route for the line is selected. A preliminary survey party runs this line, making any necessary adjustments required by circumstances encountered in the field, taking profile elevations, and gathering enough topo data in the vicinity of the line to make design of the system possible.

The system is then designed, and a plan and profile are made. Figure 6-8 shows a storm sewer plan and profile. The project here is the installation of 230 ft of 18-in. concrete sewer pipe (CSP), with a curb inlet (CI "A"). The computational length of sewer pipe is always given in terms of horizontal feet covered—the actual length of a section is, of course, greater than the computational length because of the slope.

The pipe in figure 6-8 is to run downslope from a curb inlet to a manhole in an existing sewerline. The reason for the distorted appearance of the curb inlet and manhole, which look much narrower than they would in their true proportions, is due to the exaggerated vertical scale of the profile. The appearance of the pipe is similarly distorted.

The pipe to be installed is to be placed at a gradient of 2.39 percent. The invert elevation of the outflowing 21-in. pipe at the manhole is 91.47 ft; that of the inflowing 18-in. pipe is to be 92.33 ft. Obviously there is a drop here of 0.86 ft. Of this drop, 0.25 ft is due to the difference in diameters; the other 0.61 ft is probably due to structural and velocity head losses.

From the invert in at the manhole the new pipe will extend 230 horizontal feet to the invert at the centerline of the curb inlet. The difference in elevation between the invert elevation at the manhole and the invert elevation at the curb inlet will be the product of 2.39 (the grade percentage) times 2.30 (number of 100-ft stations in 230 horizontal feet), or 5.50 ft. Therefore, the invert elevation at the curb inlet will be 92.33 ft (invert elevation at the manhole) plus 5.50 ft, or 97.83 ft. The invert elevation at any intermediate point along the line can be obtained by similar computation.

The plan shown in figure 6-8 is greatly simplified for the sake of clearness—it contains the bare minimum of data required for locating the new line. Plans used in actual practice usually contain more information.

The plan and profile constitute the paper location of the line. A final location survey party runs the line in the field. Where variations are required because of circumstances discovered in the field (such as the discovery of a large tree or some similar obstruction lying right on the line), the direction of the line is altered and the new line is tied to the paper location as previously described for a highway. The final location party may simply mark the location of the line and take profile elevations, or it may combine the final location survey and the stakeout (which is part of the construction

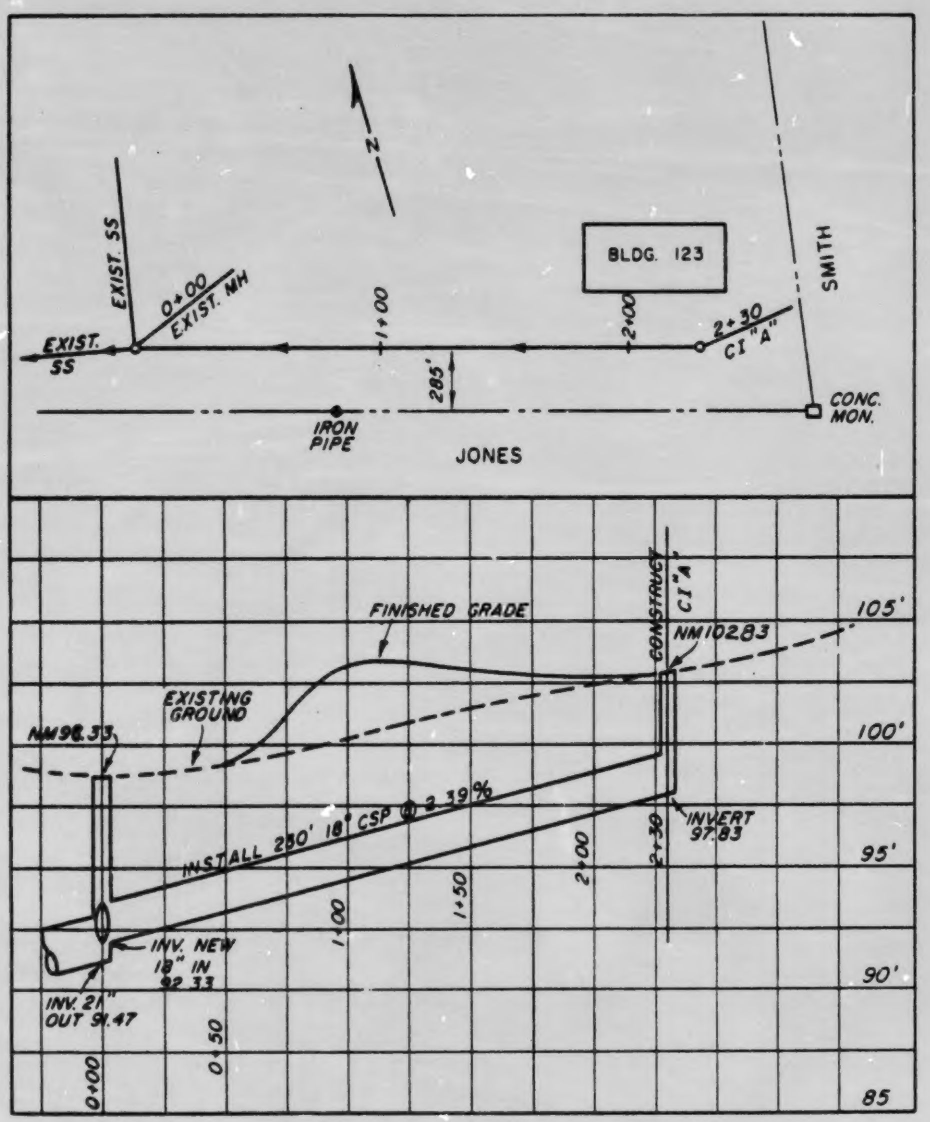

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Figure 6-8.—Storm sewer plan and profile.

rather than the route survey) in the same operation.

Sanitary Sewer Systems

A sanitary sewer, like a storm sewer, flows "downhill," and design for a sanitary system is similar to design for a storm system. Only points in which the systems differ will be mentioned here.

First, let's trace the course of a system from the outfall end up to the input end. There are still many places where the sanitary sewer outfall is simply a point where raw sewage flows into a stream, but this situation is being replaced, as rapidly as possible, by one in which inland systems outfall into sewage disposal or treatment plants. Systems in coastal communities frequently outfall into the ocean.

The outfall is, of course, the lowest point in the system, and the outfall pipe is the pipe of largest flow capacity. Following this pipe up, we come to many other pipes, branching off the main trunk in all directions, and called by various names, such as subtrunks, branch trunks, or branches. These frequently follow the courses of streams, because the course of a stream is always a downhill course.

Branching off from the subtrunks we find the street sewerlines, running along the streets on which the buildings the line serves are located. The street lines are connected to the buildings by BUILDING SEWERS, which in turn are connected to the WASTE STACKS and SOIL STACKS in the buildings. A stack is a vertical line of soil or waste piping, into which a building's soil or waste BRANCHES (pipes running from lavatory drains, shower drains, water closets, and other waste sources) convey liquid and semiliquid waste. A waste stack is one which conveys waste not containing human excrement; a soil stack is one which conveys waste containing human excrement.

Once the Q for a sanitary system has been determined, the pipe is sized and graded much as storm sewer pipe is sized and graded. A sanitary system designer is usually dealing with smaller pipe, however; and there are, of course, no storm water inlets in a sanitary system.

To help prevent clogging and to facilitate maintenance, a minimum pipe size is usually specified which may be larger than is necessary to carry the design flow at the upper ends of the system. Typical minimum sizes are 6, 8 and 10 in., and the recommended minimum velocity of sanitary sewers should be 2 ft per sec (sewer flowing full).

As the area and number of building FIXTURES (water closets, urinals, etc.) served by a system increase, the chance of all the fixtures being in use at the same time decreases; therefore, some averaging system is needed to achieve an economical design.

The design engineer bases his design on the average daily consumption of water per person in the area to be served. A typical value is 100 gals per person per day. But the use is not constant; consumption is greater in the summer than in the winter, and greater during the morning and evening than it is in the middle of the day or at night. Therefore, the average flow (based on the average consumption) is multiplied by a PEAK FLOW FACTOR to obtain the design flow. The peak flow factor is sometimes varied as the size of an area increases, because the larger the area, the greater the tendency for the flow to average out.

Typical peak flow factors might range from 4 to 6 for small areas down to 1.5 to 2.5 for larger areas. An allowance for infiltration of subsurface water into the lines is sometimes added to the peak flow to obtain the design flow. A typical infiltration allowance is 500 gals per inch of pipe diameter per mile of sewer per day.

Other Underground Utilities

More and more the advanced bases of the U.S. Armed Forces are using underground systems for distribution of water, power and communication lines. There are several reasons for this:

1. In areas subject to high winds and storms, overhead lines can present quite a problem.
2. Landing fields need a clear area without poles and overhead lines.
3. Areas used for handling and storing of materials need open spaces for cranes and other equipment.
4. In case of an enemy attack, the underground lines would be subject to far less damage compared to overhead lines.

UNDERGROUND WATERLINES.–An underground waterline flows under pressure, rather than "downhill" like a storm sewer or sanitary sewer. Therefore, the matter of gradient is not relevant to the design problem. Usually a waterline is simply placed at a uniform average distance below the surface, called under so many feet of "cover." However, it may be the case that, to avoid other utilities lines in an area, the vertical distance of a waterline below the surface may vary. When this is the case, the system is both planned and profiled; otherwise it is only planned.

As a rule there are no manholes or similar boxes in a waterline. There are usually, however, subsurface meter boxes, valve boxes, and the like. All of these are shown in the plan.

The Q for a waterline is the sum of the Q's required at the consumer end–that is, at the fixtures (such as fire hydrants and faucets) which convey water for consumption. The factors considered in determining pipe sizes include the Q's needed, the pressures desired, and the friction losses, which vary with the size, type, and length of the line. Formulas which incorporate these factors are available in engineering handbooks, and are referred to in sizing the pipe for a water system.

Water is carried from the source of supply across country by a TRANSMISSION MAIN, from which DISTRIBUTION MAINS branch off to the streets where consumer outlets exist. A building is connected to the distribution main by a BUILDING SUPPLY LINE. Fire hydrants on a street are connected directly to the distribution main. The pipe in a main is usually much larger than would be required to provide buildings with required amounts of water at desired pressure; this is because mains must be sized to provide water, not only for domestic use, but also for fire protection, industrial use, and waterfront berthing spaces. The fire demand is usually the determining factor in sizing. As a general rule, the pipe in a main serving fire hydrants is not less than 6 in.

UNDERGROUND POWERLINES.–Gradient is also, of course, not a factor in an underground powerline. One of these lines does, however, have manholes, in which adjacent sections of power cable are spliced together. Cable comes on reels, and the maximum length available on a reel is 600 ft; therefore, power manholes are usually located not more than 500 ft apart. The box on a power manhole is constructed to provide a minimum of 6 ft of head room, and usually measures 7 ft long (dimension running with the cable) by 5 ft wide.

The cable running from one manhole to the next is passed through a cylindrical, pipe-like container called a CONDUIT. The inside diameter of conduit runs from 4 in. to about 6 in. Until recently the type most frequently used was FIBER conduit (familiarly called ORANGEBURG), made of pressed wood pulp and available in 8-ft lengths, equipped with PRESSED SLEEVE COUPLINGS for joining together. Similar conduit, but made of plastic, is now coming into use. There is also IRON PIPE conduit, in 10-ft lengths with threaded joints and asbestos composition conduit (commonly refered to as TRANSITE), in 10-ft lengths with pressed sleeve couplings.

Conduit may be designed for encasement in concrete (about 3 in. minimum cover usually specified), or for laying directly in earth without encasement. A small-diameter wire is threaded through the sections, from one manhole to the next, as the conduit is laid in the trench. Later, when the cable is to be run through the conduit, the small wire is used to pull a larger PULL WIRE or PULL ROPE through. The cable is then pulled through with this one, either by hand with block-and-tackle or by a power winch on a truck.

Conduit is designed to be as watertight as possible. Still, however, there is a possibility that subsurface water may penetrate to the cable, and some water accumulates inside through condensation. Therefore, drainage must be provided for by a downslope, usually specified at a minimum of 1 percent. This may be a downslope from manhole to manhole, or it may be toward the manholes from an intermediate high point in the conduit. The bottom of a manhole is constructed to drain into a gravel bed, drain tile, or other suitable arrangement.

Conduit may be laid in a horizontal curve, from one manhole to the next, to avoid obstructions or for some other reason. Because of the fact that a curve in the conduit increases pull friction, however, the radius of such a horizontal curve must be not less than about 40 percent of the straight-line distance between the manholes. A double-slope duct (one that drains from the center toward the manholes) is laid in a vertical curve; for a curve of this kind, the radius should equal the length of the straight-line run.

Angular displacement (for curves) limits the total length of the run between manholes, because of pull friction. Graphs like the one shown in figure 6-9 give the maximum runs for various total angular displacement. Suppose, for example, that a conduit run has three horizontal displacements of 8°, 20°, and 12°, and two vertical displacements of 5° and 7°, as indicated by the chords of horizontal and vertical curves. The total angular displacement is the sum of all these, or 52°; the graph indicates that in this case the maximum conduit run would be 390 ft.

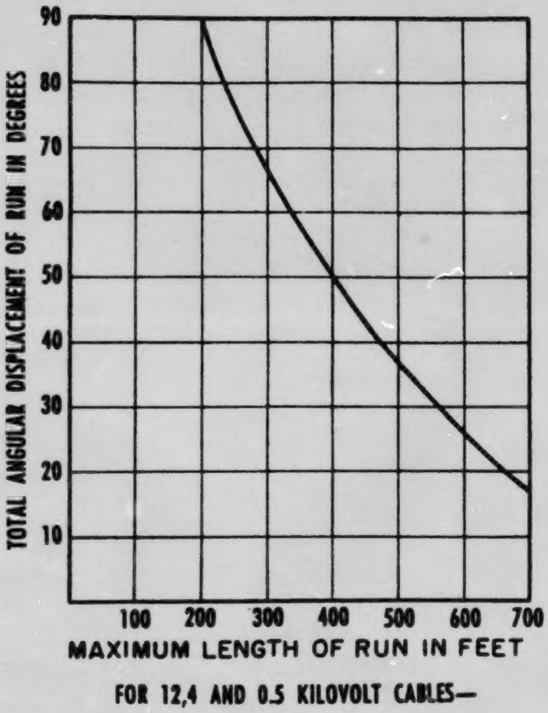

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Figure 6-9.—Maximum conduit runs for total angular displacements.

UNDERGROUND COMMUNICATIONS LINES.–Underground communications lines do not carry the high voltages of underground powerlines; therefore, the conduit for these does not need to be as substantial. Underground communications lines are usually laid in duct made of fiber, asbestos composition, or plastic.

The character of the route survey for underground water, power, or communications will depend on the circumstances. The route to be followed by the distribution part of an underground powerline will follow the streets on which the consumer outlets (buildings, street lights, traffic lights, etc.) are located–therefore, no reconnaissance or preliminary location survey is likely to be necessary. A final location survey (probably combined with stakeout) will be run from a paper location. For the transmission part of a powerline, however, reconnaissance and preliminary location surveys may be required if the line runs for a considerable distance over rough, irregular country. The same is true, generally speaking, of an underground communications line.

STAKEOUT AND AS-BUILT SURVEYS

You know that construction surveys include (1) marking points in the field to guide the crews who do the work of construction, and (2) determining the actual horizontal and/or vertical locations of points as built. We'll call these operations the STAKEOUT survey and the AS-BUILT survey, respectively.

The as-built survey consists simply of determining horizontal and vertical locations of points, and there should be little about this which you don't already know. Also, there is little to be said about building stakeout which hasn't already been said in *Engineering Aid 3 & 2*. Therefore, this section will be confined to the structures which have been mentioned in this chapter. Here again, the stakeout for an aboveground utility line (which consists simply of running a traverse and marking pole or tower locations thereon) involves nothing new to you.

There is an aspect of as-built and stakeout surveys which is of particular significance to the party chief: he must maintain close liaison with the other crews working on the project. Survey parties work independently on many types of surveys such as establishing horizontal and vertical control, running preliminary lines, shooting topo, gathering engineering data, and so on. But in stakeout, the survey party is an integral part of the construction team. Timing and scheduling are important; if line and grade stakes have not been set at the right place and at the right time, the work of entire construction crews will be delayed. The party chief must also be constantly aware of the need for replacing stakes which have been knocked out by design or accident. Frequently, changes in grade and alignment will be authorized in the field to best meet the conditions encountered. These field change orders will, in many cases, require immediate computations in the field and revisions to the stakeout. It is best to obtain, as-built data as soon as a section of the work is complete. This is particularly true if field changes have been made; the press of further construction may prevent a timely return to the job to obtain the as-built data, and users of the plans may be seriously misled in supposing that the construction conformed to the original drawings.

SEWER STAKEOUT

To stake out a sewer, you obtain data from a plan and profile which show (1) the horizontal location of each line in the system, (2) the horizontal location and character of each appurtenance, (3) the invert elevations at each appurtenance, and (4) the gradient of each line. You will also have detail drawings of each type of appurtenance. If appurtenances in the same category are of different types, you may identify them by letter symbol, as "CI "A"," and so on. In addition, identification of a particular appurtenance may be by consecutive number, as: "CI "A" #3."

The stakeout consists of setting hubs and stakes to mark the alignment and indicate the depth of the sewer. The alignment may be marked by a row of offset hubs and stakes, or by both offset hubs and a row of centerline stakes. Cuts may be shown on cut sheets (also called grade sheets or construction sheets) or may be marked on the stakes or both. The cuts shown on the centerline stakes guide the backhoe operator or ditcher operator; they are usually shown to tenths; they generally represent the cut from the surface of the existing ground to the bottom of the trench, taking into account the depth to the invert, the barrel thickness, and the depth of any sand or gravel bed. The cuts marked on the stakes next to the hubs are generally shown to hundredths and usually represent the distance from the top of the hub to the invert; these cuts guide the pipe crew. The use of these cuts in transferring the information to batter boards or various types of offset string lines was described in *Engineering Aid 3 & 2*.

If the survey party stakes only the offset hubs, then the construction crew usually sets centerline stakes for line only and uses the hubs as a guide for the depth of excavation. The extent of the stakeout and computations performed by the survey party, and the corresponding extent of such work done by the construction crew, depend on the capabilities of and the availability of personnel, and the workload. In any case, hubs and/or stakes are generally set at 25-ft intervals, though 50-ft and even 100-ft intervals have been known to suffice.

Sewer hubs are usually offset from 5 to 8 ft from the centerline. Before you enter the field, you compute from the profile the invert elevation at every station where you will set a hub. Consider figure 6-10, for example. This is a plan showing a line running from a curb inlet through two manholes to an outfall. The dotted lines are offsets (greatly exaggerated for clearness) to points where you will set hubs. Note that at stations 5 + 75 and 1 + 70.21 you set two hubs, one for the invert in and the other for the invert out.

The invert elevations at the appurtenances are given on the profile. Suppose that the invert out at CI "A" #2 is 122.87 ft. The gradient for this pipe is 2.18 percent. Station 8 + 50 lies 0.50 station from CI "A" #2, therefore, the invert elevation at station 8 + 50 is 122.87 ft minus (0.50 × 2.18), or 122.87 ft minus 1.09, or 121.78 ft. You compute the invert elevations at the other intermediate stations in the same manner.

Suppose now that you are starting the stakeout at CI "A" #2. The final location party left a centerline stake at this station. You occupy this point, turn 90 degrees left from the line to MH "A" #1, and measure off the offset—for example, 8 ft. This presumes that, if the ground slopes across the line, the high side is the side on which the hubs are placed in figure 6-10. Hubs are always placed on the high side, the reason being to prevent them from being covered by earth dozed off to form a bench for the trench-digging rig.

You drive a hub 8 ft offset from station 9 + 00, and determine the elevation of the top of the hub. The vertical distance from the top of

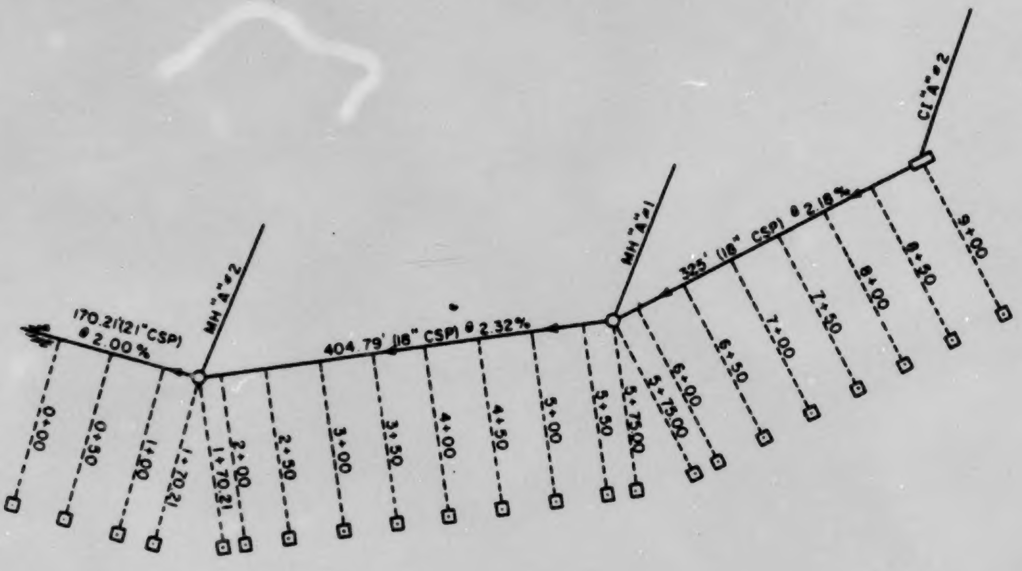

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Figure 6-10.–Sewer stakeout plan.

the hub to the invert at station 9 + 00 is the difference between the invert elevation and the elevation of the top of the hub. The invert elevation at station 9 + 00 is 122.87 ft. Suppose the elevation of the top of the hub is 126.94 ft. Then you would mark the guard stake for this hub: "CI "A" #2 inv. C 4.07'." Suppose the elevation of the top of the hub driven at station 8 + 50 was 127.33. The invert elevation at this station is 121.78; therefore, you would mark the guard stake for this station, "8 + 50, C 5.55'."

The manner in which the constructors will use these hubs to dig the trench to grade will vary according to the preference of the supervisor for one of several methods. One method involves the erection of a batter board across the trench at each hub. The top of each board is placed on the posts at a set distance above invert elevation–for example, 10 ft. Figure 6-11 illustrates this method.

Take station 9+00 in figure 6-10, for example. The elevation of the top of the hub is 126.94 ft and the invert elevation is 122.87 ft. To be 10 ft above invert elevation, the top of the batter board must be placed on the post 5.93 ft above the top of the hub. To get this distance the field constructor would simply subtract the specified

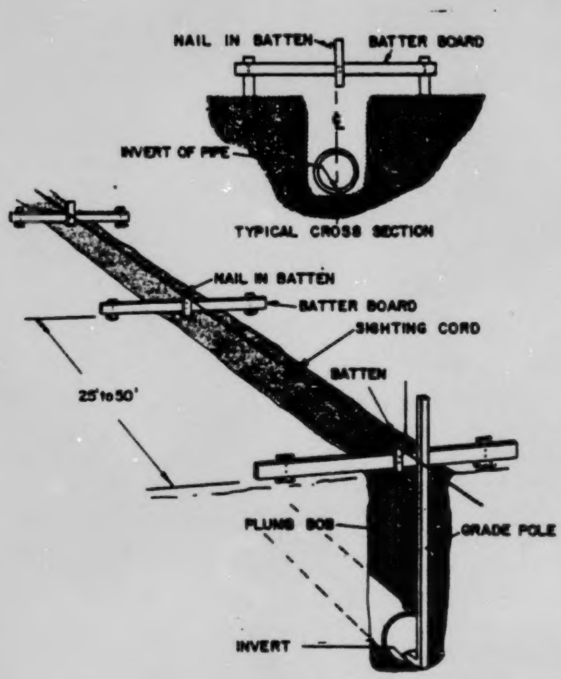

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Figure 6-11.–Setting sewerline to grade.

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cut from 10 ft. At station 8+50, for example, the height of the top of the batter board above the top of the hub would be 10 - 5.55, or 4.45 ft.

The offset is measured off from a point directly above the hub along the batter board; a mark here is directly over the center of the pipeline. Battens are nailed on the batter board to indicate sewer centerline alignment. A string is stretched and tacked along these battens; this string indicates the horizontal location of the line, and it follows the gradient of the line, but at a distance of 10 ft above the invert. The amount of cut required to be taken out at any point along the line can be determined by setting a measuring pole alongside the string. If the string indicates 8.5 ft, for example, another 1.5 ft of cut must be taken out.

Corners of rectangular appurtenance boxes are staked out much as building corners are staked out. For a box located where a line changes direction, it may be desired that the centerline of the box bisect the angle between the lines, as described for a tower. The box for a curb inlet must be exactly located with respect to a street curb to be constructed in the future; therefore, curb inlets are usually staked out with reference to the street plan rather than with reference to the sewer plan.

UNDERGROUND DUCT SYSTEM STAKEOUT

The stakeout for an underground powerline is similar to that for a sewer. For the ducts, cuts are measured to the elevation prescribed for the bottom of the duct, plus the thickness of the concrete encasement, if any. In an underground power system, the bottom of the manhole is usually about 2 ft below the bottoms of the incoming and outgoing ducts. Power and communications manholes are often combined; figure 6-12 shows plan and section views of a standard combination power and communications manhole.

Conduit and cable connections to buildings, street lighting systems, traffic light systems, and the like, are low-voltage SECONDARY lines. Duct connections from main-line manholes run to small subsurface openings called HANDHOLES on the secondary line. The handhole contains connections for takeoff to the consumer outlet. Figure 6-13 shows plan and section views of a handhole.

AIRFIELD SURVEYS

Airfield construction is of a special kind; for this reason, it is discussed here under separate heading.

AIRFIELD TERMINOLOGY

It is advisable at first to present a list of definitions of some of the terms frequently used in this highly specialized work. Figure 6-14 is a plan view of a small advanced-base field. A field of this type is constructed for operational use in a combat area; it contains a minimum of servicing facilities and is not intended for permanent occupancy. Some of the terms shown are defined as follows:

APPROACH ZONE is a trapezoidal area established beyond the end zone at each end of a runway. The approach zone must be free of obstructions on the plane of a specific GLIDE ANGLE.

APRON is a stabilized, paved or metal plank-surfaced area, designed for the temporary parking of aircraft other than at hardstands. Aprons are classified as SERVICE, WARM-UP, and PARKING.

END ZONE is a cleared and graded area that extends beyond each end of the runway. The dimensions of the end zone depend upon the safety clearances specified by the design criteria for advanced base airfields.

GLIDE ANGLE is the angle between the flight path of an airplane during a glide for landing or takeoff and a horizontal plane fixed relative to the runway. The glide angle is measured from the outer edge of the end zone.

HARDSTAND is a stabilized, paved or metal plank-surfaced parking area, of sufficient size and strength to accomodate a limited number of aircraft. Hardstands are usually dispersed over the ground area beyond the safety clearance zones of a landing strip. They provide protection for aircraft on the field by dispersal, concealment, and revetment.

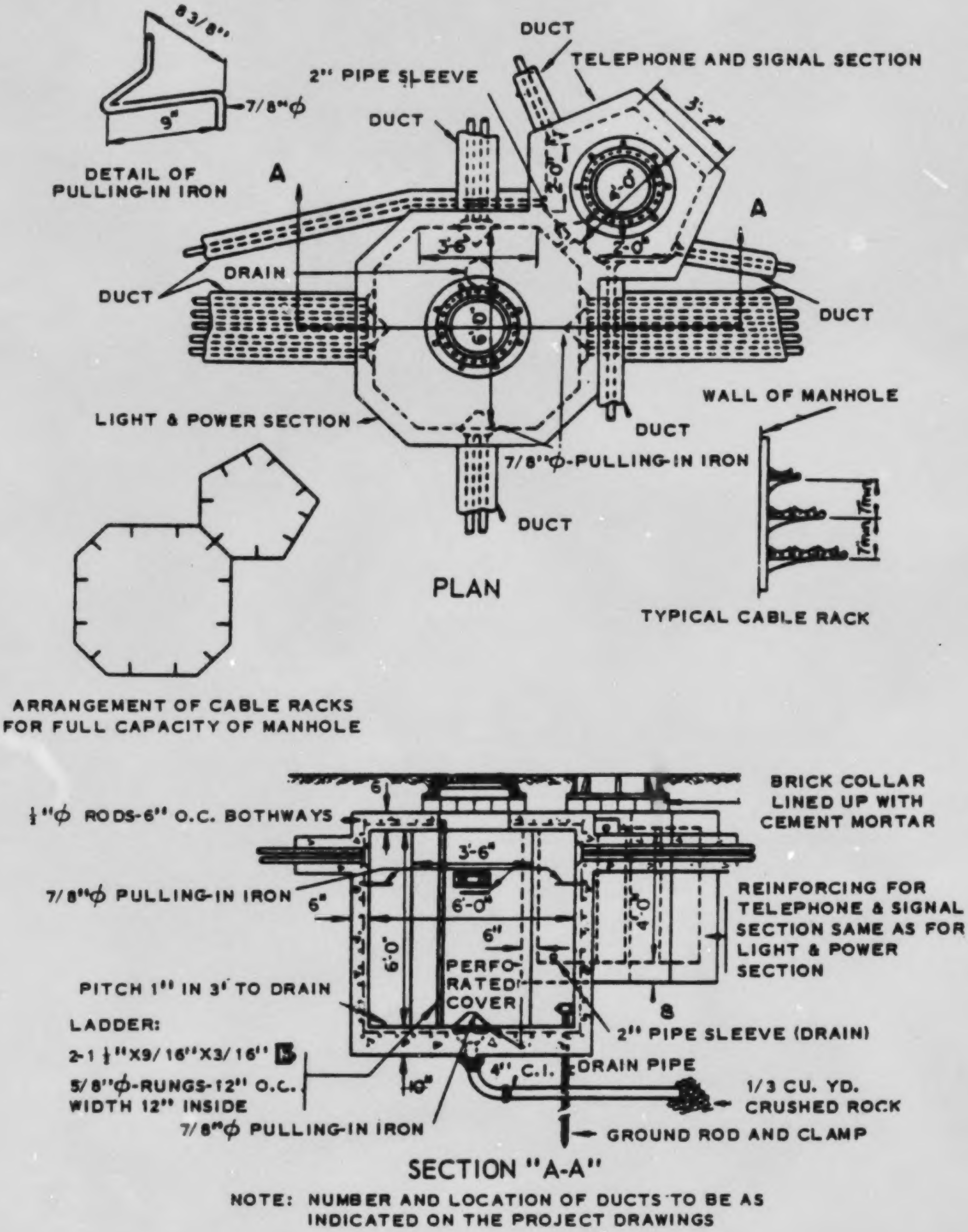

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Figure 6-12.–Standard combination power and communication manhole.

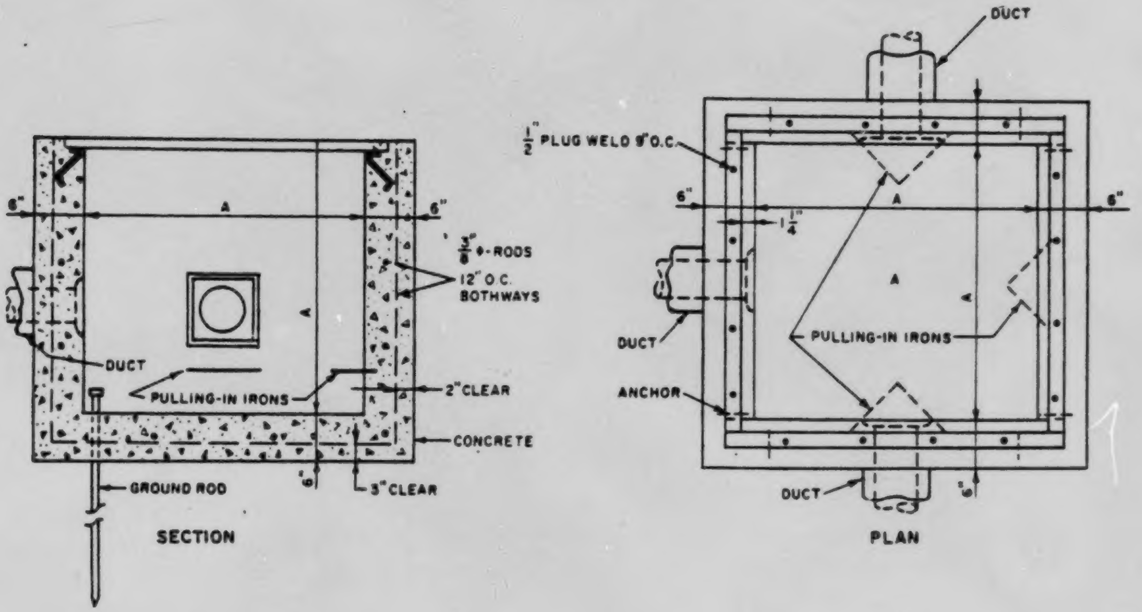

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Figure 6-13.—Handhole.

LANDING AREA is the paved portion, or runway, of the landing field. The landing area should have unobstructed approaches and should be suitable for the safe landing and takeoff of aircraft under ordinary weather conditions.

LANDING STRIP includes the landing area, the end zones, the shoulders, and cleared areas.

REVETMENT is a protective pen usually made by excavating into the side of a hill or by constructing an earth, timber, sandbag, or masonry traverse around the hardstands. Such pens provide protection against bomb fragments from high-altitude bombing but provide little protection against ground strafing. They may actually draw this type of fire if not well concealed.

RUNWAY is that portion of the landing strip, usually paved, where the aircraft actually land and take off.

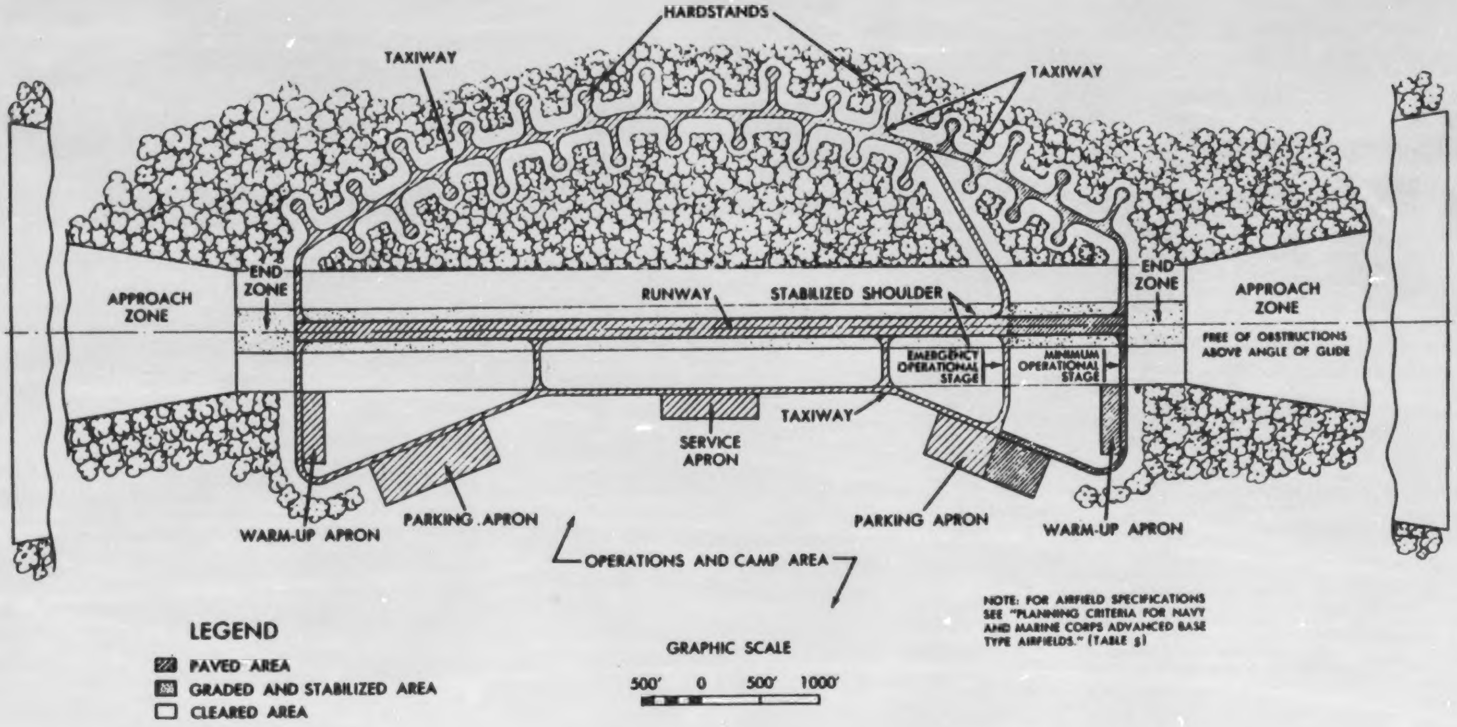
SHOULDER is the graded and stabilized area adjacent to the runway or taxiway. Although it is made capable of supporting aircraft and auxiliary equipment (such as crash trucks) in emergencies, its principal function is to facilitate surface drainage.

TAXIWAY is a specially prepared area over which aircraft may taxi to and from the landing area.

TRANSITION SURFACE is a sloping plane surface (about 1 rise to 7 run) at the edge of a landing strip. Its function is to provide lateral safety clearance for planes which accidentally run off the strip.

PLANNING AN AIRFIELD

Planning for aviation facilities requires special consideration of the type of aircraft to be accommodated; physical conditions of the site, including weather conditions, terrain, soil, and availability of construction materials; safety factors such as approach zone obstructions and traffic control; the provision for expansion; and defense. Under wartime conditions, there are also tactical considerations. All of these factors affect the number, orientation, and dimensions of runways, taxiways, aprons, hardstands, hangars, surfacing materials, and other facilities.



NOTE: FOR AIRFIELD SPECIFICATIONS SEE "PLANNING CRITERIA FOR NAVY AND MARINE CORPS ADVANCED BASE TYPE AIRFIELDS." (TABLE 5)

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Figure 6-14.—Plan view of advanced base airfield.

AIRFIELD ROUTE SURVEY

The "route" for an airfield is the horizontal location of the runway centerline; if there is more than one runway, there is, of course, more than one "route." The principal consideration with regard to the direction of a runway centerline is the average direction of the prevailing wind in the area, since planes must take off into the wind. The azimuth of the centerline will be as nearly as possible the same as the average azimuth of the prevailing wind. A study of the meteorological conditions is therefore a part of the reconnaissance survey. Other data gathered on this survey (which may be conducted on foot, by ground surface vehicle, by plane, or by all three) includes the land formation, erosional markings, vegetation, configuration of drainage lines, flight hazards, approach zone obstructions, and soil types.

From the reconnaissance data, one or more preliminary centerlines are selected for location by preliminary survey. For quick preliminary stakeout there may be two parties, working away from station 0 + 00 located at the approximate midpoint of the centerline. In a case of this kind, stations along the azimuth may be designated as plus and those along the back azimuth as minus.

Level parties follow immediately behind the transit parties, taking profile levels and cross-sections extending the width of the strip, plus an overage for shoulders and drainage channels. From the preliminary survey data, a plan and profile are made of each tentative location, and from these a selection of a final location is made.

AIRFIELD STAKEOUT

Airfield runways, taxiways, hardstands, and aprons are staked out much as a highway is staked out. There are, however, certain special considerations applying to approach zones.

You recall that an approach zone is a trapezoidal area beyond the end zone at each end of a runway, to be free of obstruction on a specific glide angle. The size of the approach zone depends on the type and stage of development of the field—for permanent naval air stations the trapezoidal area might be 10,000 ft long, with a width of 1,500 ft at the outer end. For purposes of explanation only, we'll assume that these are the dimensions of the approach zone for which you are surveying.

The glide angle for most types of aircraft is 2 percent, usually given as 50:1, or a rise (or drop) of 1 vertical for 50 horizontal. Figure 6-15 shows, in plan, profile, and isometric, an approach zone and its adjacent transition surfaces and end of runway. You must stake out this approach zone and check it for clearance, by the following procedure:

Figure 6-16 shows the approach zone in plan. The dotted line BC lies 750 ft from the centerline. The angle at B can be determined by solving the triangle CBD; tan B = 1250/10,000, or 0.125000; therefore, angle B measures 7°7'30". Determining the distance from the dotted line to the edge of the approach zone at any station is similarly a simple right-triangle solution. Suppose that AB is located at station 0 + 00. Then at station 1 + 00 the distance from the dotted line to the edge of the approach zone is 100 tan 7°7'30", or 100 (0.125), or 12.5 ft. Therefore, the distance between the centerline and the edge of the approach zone at this station is 750 + 12.5, or 762.5 ft.

To check for obstructions, you must set up a transit at the narrow end of the approach zone, set the telescope at a vertical angle equal to that which the glide plane makes with the horizontal, and take observations over the whole approach zone as indicated in figure 6-17. Determining the vertical angle is a simple right-triangle solution. If the glide angle is 50:1, then the tangent of the vertical angle is 1/50, or 0.020000, and the angle measures 1°8'50".

Figure 6-17 shows how the exact vertical location of the glide plane varies with the character of the surface of the end zone.

WATERFRONT SURVEYS

Under some circumstances it is possible to chain distances over the water; however, it is usually more convenient to triangulate offshore distances from a shore base line. No matter how you get offshore distances, however, offshore points cannot be marked like ground points with hubs or stakes. Therefore, in the location of

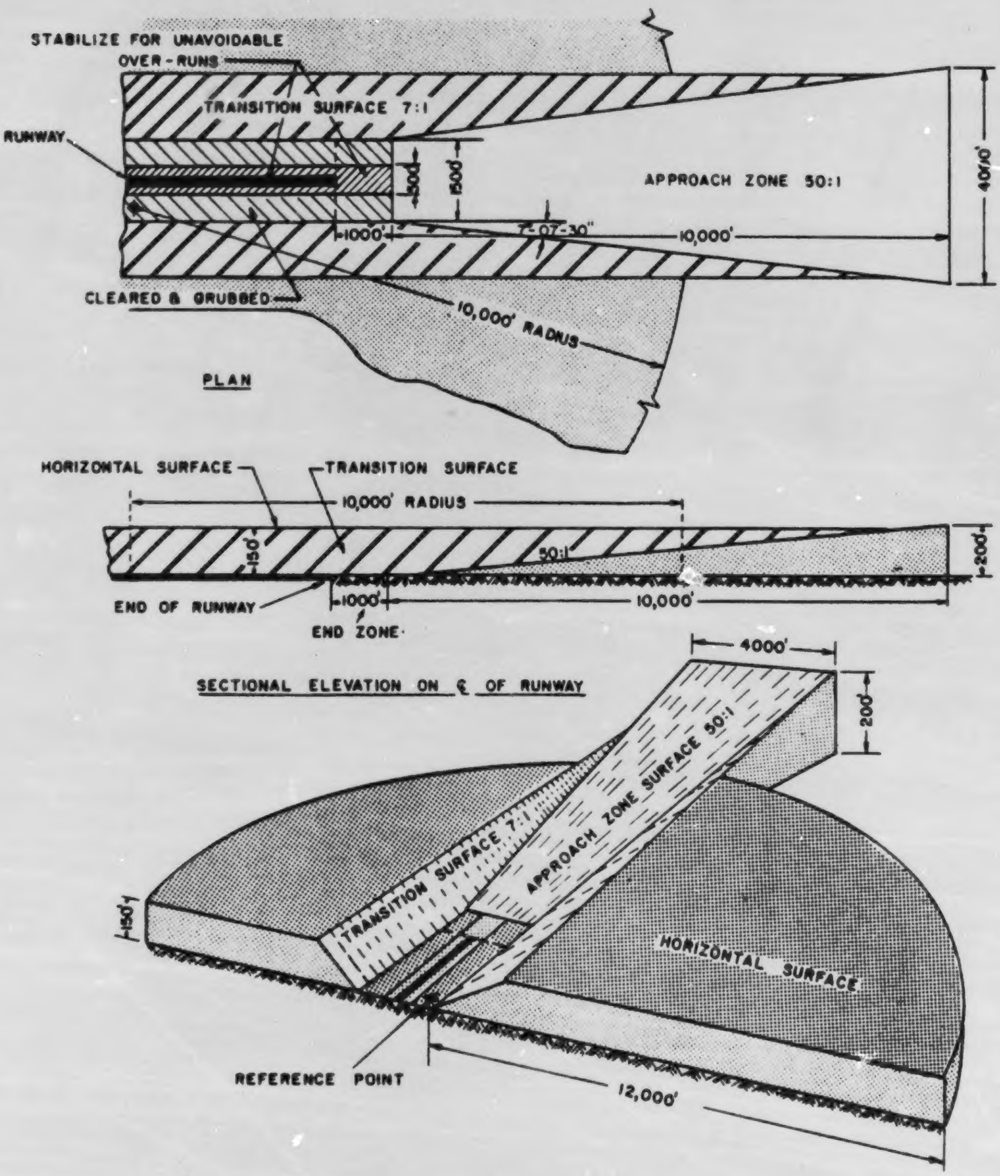

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Figure 6-15.–Runway approach zone.

offshore points there must usually be coordination between a survey party on the beach and a party afloat.

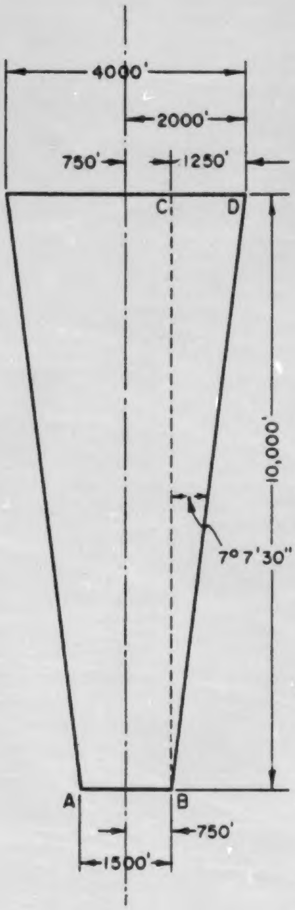

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Figure 6-16.—Plan view of approach zone.

OFFSHORE LOCATION BY CHAINING

Figure 6-18 shows a situation in which offshore locations of piles for a wharf were determined by chaining. We'll call each series of consecutive piles running offshore a "line" and each series running parallel to the shore a "row." Alignment for each line was obtained by transit, set up on a shore base line offset from the inboard row of piles. In each line the distance from one pile to the next was chained, as shown.

In figure 6-18 the lines are perpendicular to the base line, which means that the angle turned from the base line was 90° and the distance from one transit setup to the next was the same as the prescribed distance between lines. If the lines were not perpendicular to the base line, both the angle turned from the base line, the distance from one transit setup to the next, and the distance from the base line to the first offshore pile in each line would have to be determined.

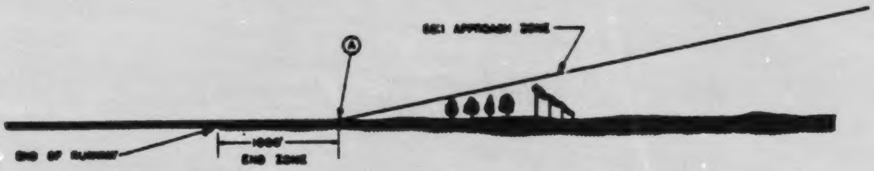
Consider figure 6-19, for example. Here the angle between each line and the base line (either as prescribed or as measured by protractor on a plan) is 60°40'. You can determine the distance between transit setups by solving the triangle JAB for AB, JA being drawn from transit setup B perpendicular to the line from transit setup A through piles 1, 2, 5, 10, 16, and 25. AB measures 50/sin 60°40', or 50/0.87178, or 57.35 ft. This, then, is the distance between adjacent transit setups on the base line.

The distance from the base line to the first offshore pile in any line may also be determined by right-triangle solution. For pile #1 this distance is prescribed as 50 ft. For piles 2, 3, and 4, first solve the triangle A2L for 2L, which is 100/tan 29°20', or 100/0.56193, or 177.95 ft. The distance from 2 to Q is 150 ft; therefore, QL measures 177.95 - 150, or 27.95 ft. QD amounts to 27.95/tan 60°40', or 27.95/1.77955, or 15.71 ft. Therefore, the distance from transit setup D to pile #8 is 50 + 15.71, or 65.71 ft. Knowing the length of QL and the distance from 3 to Q, you can determine the distance from 3 to Q, you can determine the distance from setup point B to pile 3 by solving the right triangle LB3 for B3.

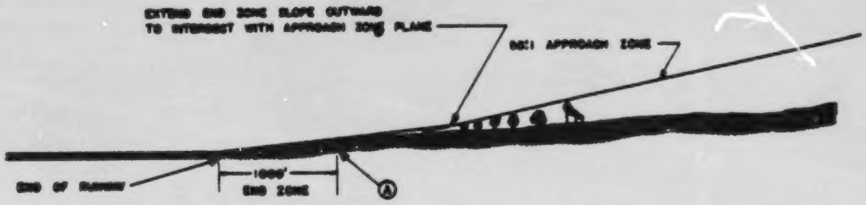
You can determine the distance E9 by solving the right triangle M5A and proceeding as before. You can determine the distance F15, G22, and H23 by solving the right triangle AN10 and proceeding as before. For pile #24 the distance I24 amounts to 50 tan 29°20', or 50(0.56193), or 28.10 ft.

OFFSHORE LOCATION BY TRIANGULATION

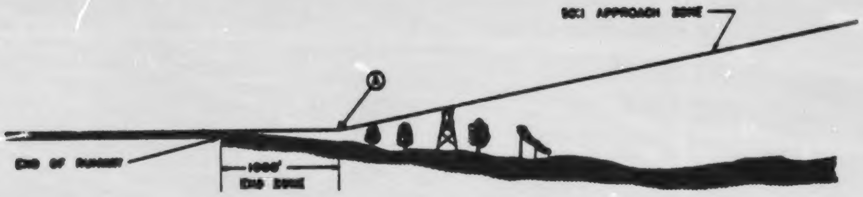
For piles located farther offshore, the triangulation method of location is preferred. A pile



LEVEL END ZONE (NO LONGITUDINAL SLOPE)


EXTEND END ZONE SLOPE OUTWARD
TO INTERSECT WITH APPROACH ZONE PLANE
50:1 APPROACH ZONE
END OF RUNWAY
1000'
END ZONE
A

ASCENDING END ZONE LONGITUDINAL SLOPE



DESCENDING END ZONE LONGITUDINAL SLOPE

82.32

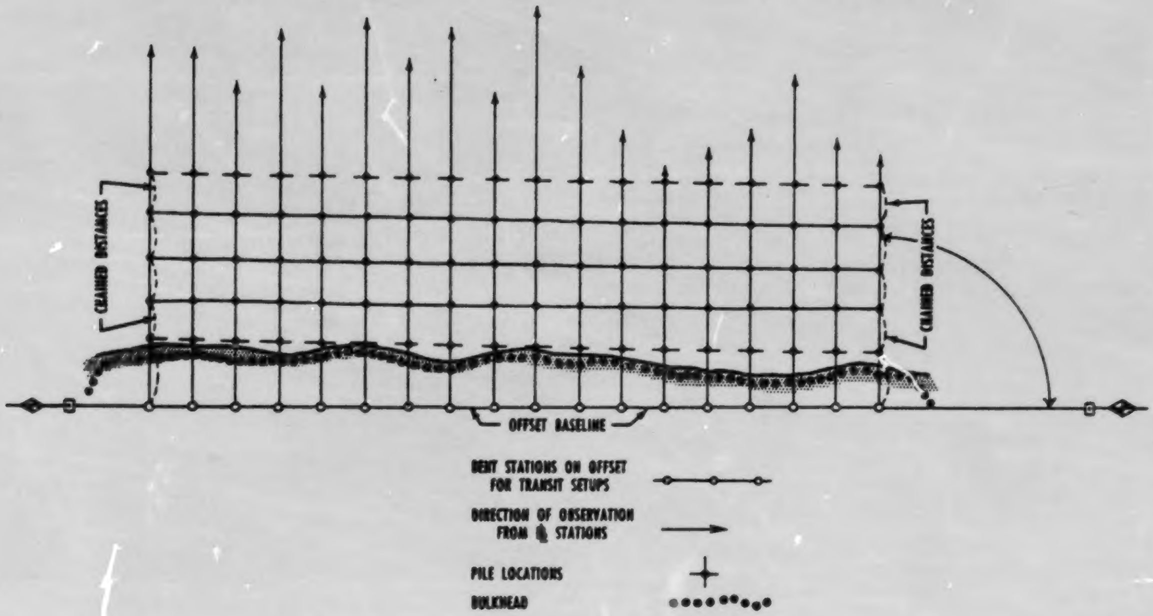
Figure 6-17.–Approach clearance for different types of end zones.

location diagram is shown in figure 6-20. It is presumed that the piles in section X will be located by the method just described, while those in section Y will be located by triangulation from the two control stations shown.

The base line measures (1038.83 - 443.27), or 595.56 ft, from control station to control station. The middle line of piles runs from station 7 + 41.05, making an angle of 84° with the base line. The piles in each bent are 10 ft apart; bents are identified by letters and piles by numbers. The distance between adjacent transit setups in the base line is 10/sin 84°, or 10/0.994522, or 10.05 ft.

Bents are located 20 ft apart. The distance from the centerline base line transit setup at station 7 + 41.05 to pile #3 is 70 ft. The distance from station 7 + 51.10 to pile #2 is 70 + 10 tan 6°, or 70 + 10(0.105104), or 70 + 1.05, or 71.05 ft. The distance from station 7 + 61.15 to pile #1 is 71.05 + 1.05, or 72.10 ft. The distance from station 7 + 31.00 to pile #4 is 70-1.05, or 68.95 ft; that from station 7 + 20.95 to pile #5 is 68.95 - 1.05, or 67.90 ft.

You can determine the angle you turn, at a control station, from the base line to any pile location, by triangle solution. Consider pile #61, for example. This pile is located (240 + 72.10), or 312.10 ft from station 7 + 61.15 on the base line. Station 7 + 61.15 is located (1038.83 - 761.15), or 277.68 ft from control station 10 + 38.83. The angle between the line from station 7


CHAINED DISTANCES
CHAINED DISTANCES
OFFSET BASELINE
BENT STATIONS ON OFFSET FOR TRANSIT SETUPS
DIRECTION OF OBSERVATION FROM ℄ STATIONS
PILE LOCATIONS
BULKHEAD

82.33

Figure 6-18.—Offshore location by chaining.

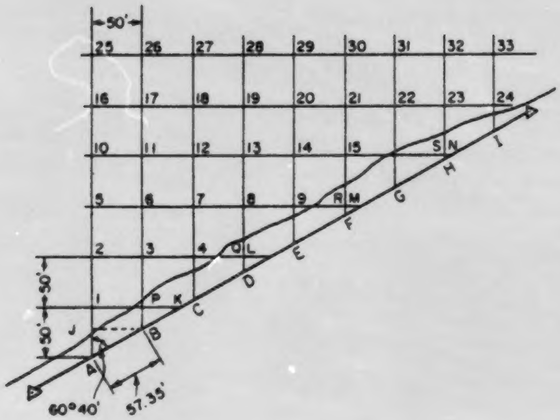


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Figure 6-19.—Offshore locations in line oblique to the base line.

+ 61.15 through pile #61 and the base line measures (180° - 84°), or 96°. Therefore, you are dealing with the triangle shown in figure 6-21. You want to know the size of angle A.

First solve for b by the law of cosines, in which $b^2 = a^2 + c^2 - 2ac \cos B$, as follows:

$$b^2 = 312.10^2 + 277.68^2 - 2(312.10)(277.68) \cos B$$

The cosine of an angle larger than 90° is the same as minus the cosine of its supplement; therefore, the cosine of 96° is the same as minus the cosine of 84°, or - 0.10453. So now we have:

$$b^2 = 312.10^2 + 277.68^2 - 2(312.10)(277.68)(-0.10453)$$

$$b^2 = 97406.41 + 77106.18 + 18117.96$$

$$b^2 = 192630.55$$

$$b = \sqrt{192630.55} = 438.89 \text{ ft.}$$

Knowing the length of b, you can now determine the size of angle A by the law of sines. Sin A = 312.10 sin 84°/438.89, or 312.10 (0.99452)/438.89, or 0.70699. This means that angle A measures, to the nearest minue, 45°.

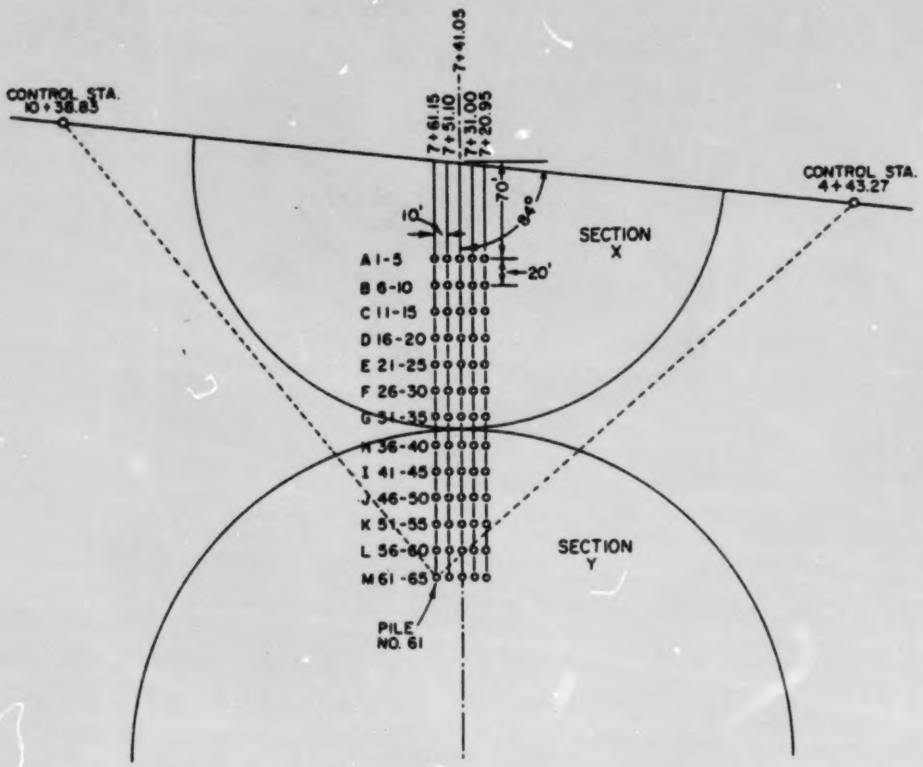

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Figure 6-20.–Pile location diagram.

To determine the direction of this pile from control station 4 + 43.27, you would solve the triangle shown in figure 6-21. The length of side c equals the distance along the base line from control station 4 + 43.27 to station 7 + 61.15; the length of side a equals the distance from station 7 + 61.15 to pile #61. You would solve for side b as follows:

$$b^2 = 312.10^2 + 317.88^2 - 2(312.10)(317.88)(\cos 84^\circ)$$
$$b^2 = 97406.41 \quad 101047.69 - (198357.12)(0.104523)$$
$$b^2 = 198454.10 - 20733.87$$
$$b^2 = 177720.23$$
$$b = \sqrt{177720.23} = 421.56 \text{ ft.}$$

You would solve for angle D as follows:

$$\sin D = \frac{312.10 \sin 84^\circ}{421.56} = \frac{312.10(0.99452)}{421.56} = 0.73646$$

Angle D, then, would measure 47°26′. It would probably be necessary to locate in this fashion only the two outside piles in each bent; the piles between these two could be located by measuring off the prescribed spacing on a tape stretched between the two. For the direction from control station 10 + 38.83 to pile #65 (the other outside pile in bent M) you would solve the triangle shown in figure 6-22 as follows:

$$b^2 = 307.90^2 + 317.88^2 - 2(307.90)(317.88)(-0.10453)$$
$$b^2 = 94802.41 + 101047.69 + 20461.80$$
$$b^2 = 216311.90$$
$$b = \sqrt{216311.90} = 465.09 \text{ ft.}$$

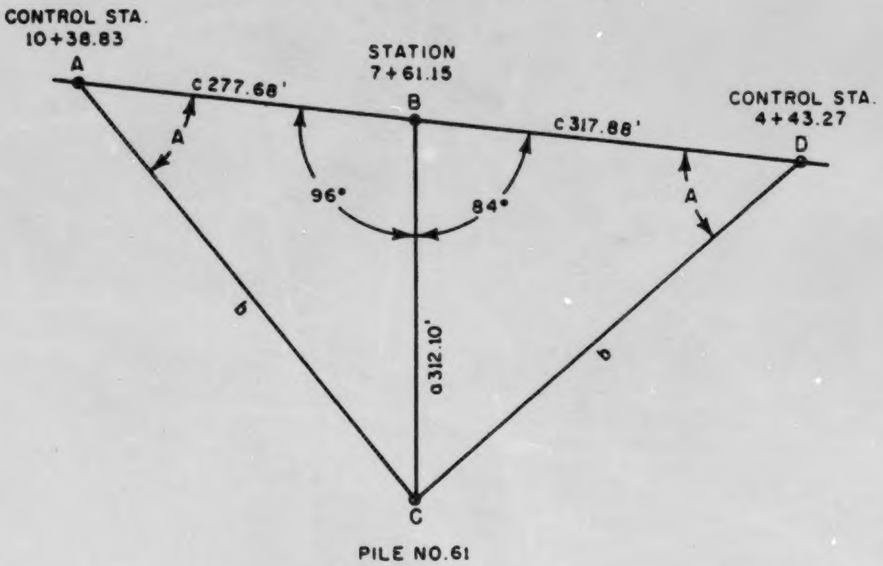

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Figure 6-21.—Trigonometric solution for pile #61.

$$\sin A = \frac{307.90(0.994522)}{465.09} = 0.658388$$

Angle A = 41°10′

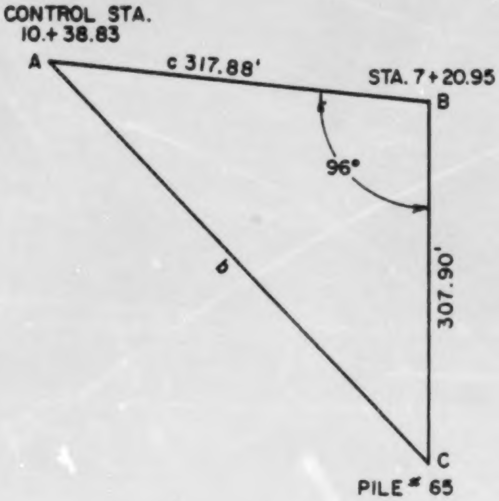

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Figure 6-22.—Trigonometric solution for pile #65.

For each control station a PILE LOCATION SHEET like the one shown in figure 6-23 would be made up. If desired, the direction angles for the piles between #61 and #65 could be computed and inserted in the intervening spaces.

DREDGING SURVEYS

The excavation of material in underwater areas is called DREDGING, and a DREDGE is an excavator afloat on a barge. A dredge may get itself into position by cross-bearings, taken from the dredge on objects of known location on the

TRANSIT AT CONTROL STA. 10+38.83				
BENT	PILE #	BS STATION	ANGLE FROM BS	REMARKS
M	61	7+61.15	45° 00'	∠ RIGHT
	62			
	63			
	64			
	65	7+61.15	41° 10'	∠ RIGHT

82.39

Figure 6-23.—Pile location sheet.

beach, or by some other piloting method. Many times, however, dredges are positioned by survey triangulation. The method of determining direction angles from base line control points is the same as that just described.

EARTHWORK COMPUTATIONS

The computation of earthwork volumes is a feature in nearly all construction surveys—especially in highway and airfield construction. The computation of earthwork for airfield construction is similar to that of a highway. The earthwork procedures for highways were discussed in *Engineering Aid 3 & 2*; the computation of volumes by the average-end-area method, the contour method, and the prismoidal method were explained.

A highway designer's concern is economy on earthwork. He wants to know exactly where, how far, and how much earth to move in a section of road. The ideal situation is to balance the cut and fill and limit the haul distance. The technique for balancing cut and fill, and determining the economical haul distance, is by the MASS DIAGRAM method.

MASS DIAGRAM METHOD

The mass diagram is a graph or curve on which the algebraic sums of cuts and fills are plotted against linear distance. Before these cuts and fills are tabulated, the swells and compaction factors are considered in computing the yardage. Earthwork that is in-place will yield more yardage when excavated and less yardage when being compacted. An example of this is sand: 100 cubic yards in-place yields 111 cubic yards loose and only 95 cubic yards when compacted. See table 6-2 of soil conversion factors. These factors should be used when preparing a table of "cumulative yardage" for a mass diagram. Cuts are indicated by a rise in the curve, and are considered positive; fills are indicated by a drop in the curve, and are considered negative. The yardage between any pair of stations can be determined by inspection. This feature makes the mass diagram a great help in the attempt to balance cuts and fills within the limits of economic haul.

The limit of economic haul is reached when the cost of haul and the cost of excavation become equal. Beyond that point it is cheaper to waste the cut from one place, and to fill the

Table 6-2.—Soil Conversion Factors (Conversion Factors for Earth-Volume Change)

Soil Type	Soil condition initially	Converted to		
		In-place	Loose	Compacted
Sand	In-place	1.00	1.11	0.95
	Loose	.90	1.00	.86
	Compacted	1.05	1.17	1.00
Loam	In-place	1.00	1.25	0.90
	Loose	.80	1.00	.72
	Compacted	1.11	1.39	1.00
Clay	In-place	1.00	1.43	0.90
	Loose	.70	1.00	.63
	Compacted	1.11	1.59	1.00
Rock (blasted)	In-place	1.00	1.50	1.30
	Loose	.67	1.00	.87
	Compacted	.77	1.15	1.00
Hard coral	In-place	1.00	1.50	1.30
	Loose	.67	1.00	.87
	Compacted	.77	1.15	1.00

143.117

adjacent hollow with material taken from a nearer borrow pit. The limit of economic haul will, of course, vary at different stations on the project, depending on the nature of the terrain, the availability of equipment, the type of material, accessibility, availability of manpower, and other considerations.

There exists what is called a FREE-HAUL distance–that is, a distance over which it is considered that haul involves no extra cost. This distance is usually taken to be about 500 ft–meaning that it is only for hauls longer than 500 ft that the limits of economic haul need to be considered.

Tabulating Cumulative Yardage

The first step in making a mass diagram is to prepare a TABLE OF CUMULATIVE YARDAGE like the one shown in table 6-3. Under "End Areas"–you put the cross-section area at each station–sometimes this is cut, sometimes fill, and sometimes (as at station 9 + 00 and 15 + 00) part cut and part fill. Under "Volumes" you put the volumes of cut and/or fill between stations, computed from the average end areas and the distance between sections, in cubic yards. Note that, besides the sections at each full station, sections are taken at every plus where both the cut and the fill are zero. Note also that cut volumes are designated as plus, fill volumes as minus.

Under "Algebraic sums volumes, cumulative" you put the cumulative volume at each station and each plus, computed in each case by determining the algebraic sum of the volume at that station or plus and the preceding cumulative total. For example: at station 8 + 00 the cumulative total is - 563. At station 9 + 00 there is a volume of cut of + 65 and a volume of fill of - 305, making a net of - 240. The cumulative total at station 9 + 00, then, is (-563) + (-240), or -803.

Plotting Mass Diagram

Figure 6-24 shows the values from the table of cumulative yardage plotted on a mass diagram. The vertical coordinates are cumulative volumes, plus or minus from a LINE OF ZERO YARDAGE, each horizontal line representing an increment of 200 cu yds. The horizontal coordinates are the stations, each vertical line representing a full 100-ft station.

As you can see, the mass diagram makes it possible for you to determine, by inspection, the yardage of cut or fill lying between any pair of stations. Between station 0 + 00 and station 3 + 50, for example, there are about 800 cu yds of cut. Between station 3 + 50 and station 7 + 00 there are about 800 cu yds of fill (descending curve). Between station 7 + 00 and station 10 + 50 there are about 850 cu yds of fill (curve still descending), and so on.

Remember that sections where the volume (yardage) changes from cut to fill correspond to a maximum in the mass diagram curve, and sections where it changes from fill to cut correspond to a minimum. The peaks and the lowest points of the mass diagram, which represent the maximum or minimum yardage, occur at, or near, the gradeline on the profile.

Balancing Cuts and Fills

To understand the manner in which the mass diagram is used to balance cuts and fills and how haul limit is determined, let us examine figure 6-24. Here the profile of a road, stations 0 + 00 to 20 + 00, has been plotted above the mass diagram. You can see that they are plotted on the same horizontal scale. The labeled sections and arrows on the profile show relatively what is to be done to the cuts and fills, and where the limit of economical haul is exceeded, the cut is wasted, and the fill is borrowed.

In figure 6-24, a 500-ft haul limit line has been inserted into the mass diagram curve above and below the lines of zero yardage (the 500-ft distance is laid out to scale horizontally parallel to the line of zero yardage). The terminal points of these haul limit distances were projected to the profile curve as indicated. You can see that the cut lying between stations 1 + 00 and 3 + 50 can be hauled economically as far as station 6 + 00, that lying between stations 10 + 50 and 13 + 00 as far as station 8 + 00, and that lying between stations 14 + 00 and 16 + 50 as far as station 19 + 00. This leaves the cut between stations 0 + 00 and 1 + 00, the fill between stations 6 + 00 and 8 + 00, the cut between

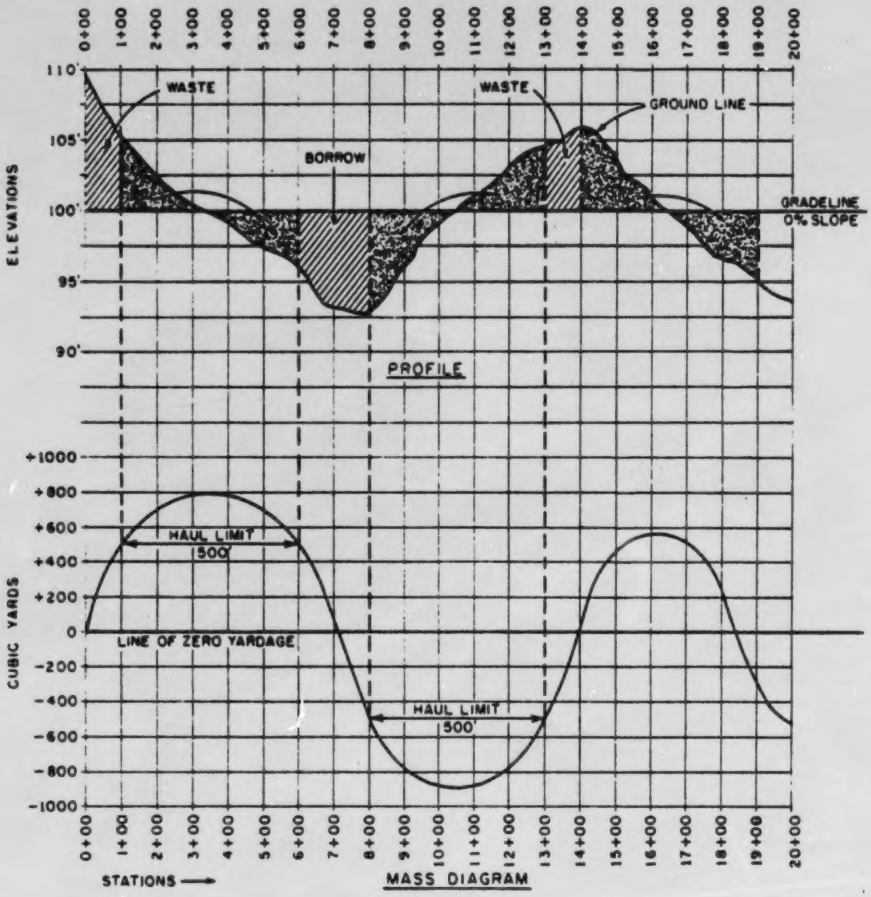

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Figure 6-24.—Profile and mass diagram.

stations 13 + 00 and 14 + 00, and the fill between stations 19 + 00 and 20 + 00.

As indicated in figure 6-24, the cut between stations 0 + 00 and 1 + 00, lying outside the limit of economical haul distance, would be wasted; that is, dumped into a nearby spoil area or ravine. The cut between stations 1 + 00 and 3 + 50 would be dumped into the adjacent fill space between stations 3 + 50 and 6 + 00. The fill space between stations 6 + 00 and 8 + 00 would be filled with borrow; that is, material taken from a nearby borrow pit. The fill space between stations 8 + 00 and 10 + 50 would be filled with the cut between stations 10 + 50 and 13 + 00, and the space between stations 16 + 50 and 19 + 00 would be filled with cut lying between stations 14 + 00 and 16 + 50. You will notice that the haul limit on the last section of the mass diagram (between stations 14 + 00 and 19 + 00) is almost on the line of zero yardage. This ~~haul~~ limit distance is also called the balance line, because the volume of cut is equal to the volume of fill. If, for example, the balance line on the last section of the mass diagram in figure 6-24 is only about 400 ft, then instead of wasting the cut between stations 13 + 00 and 14 + 00, you would use that to fill the hollow between stations 19 + 00 and 20 + 00. Surplus

Table 6-3.—Table of Cumulative Yardage

CUMULATIVE YARDAGE

STATION	END AREAS (FT^2)		VOLUMES (YD^3)		ALGEBRAIC SUMS VOLUMES, CUMULATIVE
	CUT	FILL	CUT	FILL	
0 + 00	186	0	---	---	0
1 + 00	65	0	+465	---	+465
2 + 00	44	0	+202	---	+667
3 + 00	22	0	+122	---	+789
3 + 50	0	0	+20	---	+809
4 + 00	0	22	---	-20	+789
5 + 00	0	44	---	-122	+667
6 + 00	0	65	---	-202	+465
7 + 00	0	186	---	-465	0
8 + 00	0	119	---	-563	-563
9 + 00	35	46	+65	-305	-803
9 + 08	0	0	+5	-7	-805
10 + 00	0	22	---	-37	-842
10 + 50	0	0	---	-20	-862
11 + 00	22	0	+20	---	-842
12 + 00	44	0	+122	---	-720
13 + 00	87	0	+242	---	-478
14 + 00	218	43	+563	-80	5
15 + 00	64	22	+521	-120	+406
15 + 07	0	0	+8	-8	+406
16 + 00	32	0	+55	---	+461
16 + 50	0	0	+30	---	+491
17 + 00	0	32	---	-30	+461
18 + 00	0	61	---	-172	+289
19 + 00	0	157	---	-405	-116
20 + 00	90	95	+166	-466	-416

82.47

cut remaining would naturally be wasted after allowing for shrinkage in the filled spaces.

LAND SURVEYS

Land surveying includes surveys to locate and monument the boundaries of a property; preparation of a legal description of the limits of a property and of the area included; preparation of a property map; resurveys to recover and remonument property corners; and surveys to subdivide property.

It is sometimes necessary to retrace surveys of property lines, to reestablish lost or obliterated corners, and to make ties to property lines and corners. For example, a retracement survey of property lines may be required to assure that the military operation of quarry excavation does not encroach on adjacent property where excavation rights have not been obtained. Similarly, an access road from a public highway to the quarry site which crosses privately owned property should be tied to the property lines that are crossed so that correctly executed easements can be obtained to cross the tracts of private property.

EAs may be required to accomplish property surveys at naval activities outside the continental limits of the United States for the construction of naval bases and the restoration of such properties to property owners. The essentials of land surveying as practiced in various countries are similar in principle. Although the principles pertaining to the surveys of public and private lands within the United States are not necessarily directly applicable to foreign countries, a knowledge of these principles will enable the EA to conduct the survey in a manner required by the property laws of the nation concerned.

In the United States, land surveying is a survey conducted for the purpose of ascertaining the correct boundaries of real estate property for legal purposes. In accordance with Federal and States laws, the right and/or title to landed property in the United States can be transferred from one person to another only by means of a written document, commonly called a DEED. To constitute a valid transfer, a deed must meet a considerable number of legal requirements, some of which vary in different states of the Union. In all the states, however, a deed must contain an accurate description of the boundaries of the property.

A right in real property need not be complete, outright ownership (called ownership in FEE SIMPLE.) There are numerous lesser rights, such as LEASEHOLD (right to occupancy and use for a specified term) or EASEMENT (right to make certain specified use of property belonging to someone else). But in any case, a valid transfer of any type of right in real property usually involves an accurate description of the boundaries of the property.

As mentioned previously, the EA may be required to perform various land surveys. The EA1, as survey team or crew leader, must have a knowledge of the principles of land surveys in order to plan his work accordingly.

PROPERTY BOUNDARY DESCRIPTION

A parcel of land may be described by METES AND BOUNDS; by giving the coordinates of the property corners with reference to the PLANE COORDINATES system; by a deed reference to a description in a previously RECORDED DEED; or by references to block and individual property numbers appearing on a RECORDED MAP.

By Metes and Bounds

When a tract of land is defined by giving the bearings and lengths of all boundaries it is said to be described by METES and BOUNDS. This is an age-old method of describing land and still forms the basis for the majority of deed descriptions in the eastern states of the U.S., and in many foreign lands. A good metes-and-bounds description starts at a point of beginning which should be monumented and referenced by ties or distances from well established monuments or other reference points. The bearing and length of each side is given in turn around the tract to close back on the point of beginning. Bearing may be true or magnetic grid, preferably the former. When magnetic bearings are read, the declination of the needle and the date of the survey should be stated. The stakes or monuments placed at each corner should be described to aid in their recovery in the future. Ties from corner monuments to witness points (trees, poles, boulders, ledges, or other semipermanent or permanent objects) are always helpful in relocating corners, particularly where the corner markers themselves lack permanence. In timbered country, blazes on trees on or adjacent to a boundary line are most useful in reestablishing the line at a future date. It is also advisable to state the names of abutting property owners along the several sides of the tract being described. Many metes-and-bounds descriptions fail to include all of these particulars and are frequently very difficult to retrace or locate in relation to adjoining ownerships.

One of the reasons why the determination of boundries in the U.S. if often difficult is the fact that early surveyors often confined themselves to MINIMAL descriptions—that is, to a bare statement of the METES AND BOUNDS, COURSES AND DISTANCES. Nowadays good practice requires that a land surveyor include all relevant information in his description.

In preparing the description of a property, the surveyor should bear in mind that the description must clearly identify the location of the property and must give all necessary data from which the boundaries can be reestablished at any future date. The written description contains the greater part of the information shown on the plan. Usually both a description and a plan are prepared and, when the property is transferred, are recorded according to the laws of the county concerned. The metes-and-bounds description of the property shown in figure 6-25 is given below.

"All that certain tract or parcel of land and premises, hereinafter particularly described, situate, lying and being in the Township of Maplewood in the County of Essex and State of New Jersey and constituting lot 2 shown on the revised map of the Taylor property in said township as filed in the Essex County Hall of Records on March 18, 1944."

"Beginning at an iron pipe in the northwesterly line of Maplewood Avenue therein distant along same line four hundred and thirty-one feet and seventy-one one-hundredths of a foot northeasterly from a stone monument at the northerly corner of Beach Place and Maplewood Avenue; thence running (1) North Forty-four degrees thirty-one and one-half minutes West along land of H. L. Coombs one hundred and fifty-six feet and thirty-two one-hundredths of a foot to an iron bar; thence turning and running (2) North forty-five degrees twenty-eight and one-half minutes East along land of S. M. Taylor eighty-seven feet to an iron bar; thence turning and running (3) South forty-four degrees and thirty-one and one-half minutes East along land of B. A. Toler one hundred and fifty-six feet and thirty-two one-hundredths of a foot to an iron bar in a north-westerly line of Maplewood Avenue; thence turning and running (4) South forty-five degrees twenty- eight and one-half minutes West along said line of Maplewood Avenue eighty-seven feet to the point and place of beginning; all bearings being true and the lot containing a calculated area of thirteen thousand six hundred square feet. This description has been prepared from a survey made by R.F. Jones, Licensed Land Surveyor, New Jersey

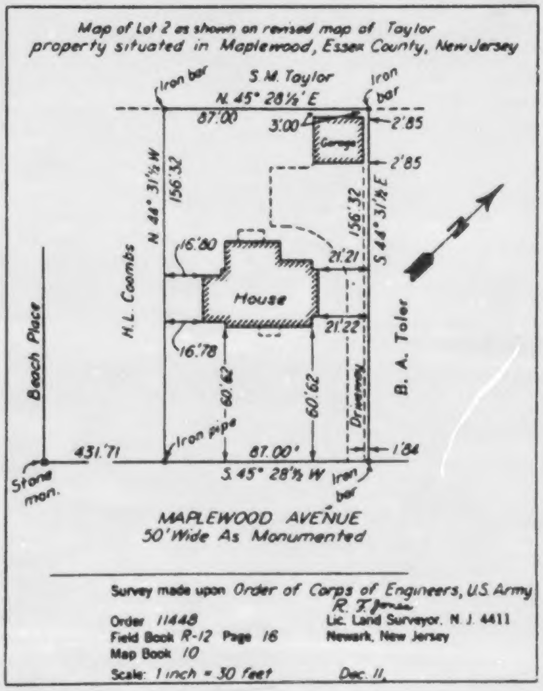

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Figure 6-25.—Lot plan by metes and bounds.

No. 4411, said survey being dated December 11, 1944."

Another form of a lot description may be presented as follows:

"Beginning at the northeasterly corner of the tract herein described; said corner being the intersection of the southerly line of Trenton Street and the westerly line of Ives Street; thence running S 6°29′54″ E bounded easterly by said Ives Street, a distance of two hundred and twenty-seven one hundredths (200.27) feet to the northerly line of Wickenden Street; thence turning an interior angle of 89°59′16″ and running S 83°30′50″ W bonded southerly by said Wickenden Street, a distance of one hundred and no one hundredths (100.00) feet to a corner; thence turning an interior angle of . . . etc."

You will notice that in the above example, interior angles were added to the bearings of the boundary lines, which will be another help in retracing lines.

By Rectangular System

In the early days (from 1785) of the United States, provisions were made to subdivide territorial lands into townships and sections thereof, along lines running with the cardinal directions of north-south, east-west. Later, as additional lands were added to the public domain, such lands were subdivided in a similar manner.

However, these methods of subdividing lands do not apply in the eastern seaboard (original 13 states) and in Hawaii, Kentucky, Tennessee, Texas and West Virginia. For laws regulating the subdivision of public lands and the recommended surveying methods, check the instruction manual published by the Bureau of Land Management, Washington, D.C.

By Plane Coordinates

For many years the triangulation and traverse monuments of various domestic and foreign survey agencies have been defined by their geographic positions, that is, by their latitudes and longitudes. Property corners might be definitely fixed in position in the same way. The necessary computations are involved and too few land surveyors are sufficiently well versed in the theory of geodetic surveying for this method to attain widespread use. In recent years, plane coordinate systems have been developed and used in many states and in many foreign countries. These grid systems involve relatively simple calculations and their use in describing parcels of land is increasing. Every state in the American Union is now covered by a statewide coordinate system commonly called a GRID SYSTEM.

As with any plane-rectangular coordinate system, a projection employed in establishing a State coordinate system may be represented by two sets of parallel straight lines, intersecting at right angles. The network thus formed is the GRID. A system of lines representing geographic parallels and meridians on a map projection is termed GRATICULE. One set of these lines is parallel to the plane of a meridian passing approximately through the center of the area shown on the grid, and the grid line corresponding to that meridian is the Y-AXIS of the grid. The Y-axis is also termed the CENTRAL MERIDIAN of the grid. Forming right angles with the Y-axis and to the SOUTH of the area shown on the grid is the X-AXIS. The point of intersection of these axes is the ORIGIN of coordinates. The position of a point represented on the grid can be defined by stating two distances, termed COORDINATES. One of these distances, known as the X-COORDINATE, gives the position in an EAST-and-WEST direction. The other distance, known as the Y-COORDINATE, gives the position in a NORTH-and-SOUTH direction; this coordinate is always positive. The X-coordinates increase in size, numerically, from west to east; the Y-coordinates increase in size from south to north. All X-coordinates in an area represented on a State grid are made positive by assigning the origin of the coordinates: X = 0 plus a large constant. For any point, then, the X-coordinate equals the value of X adopted for the origin, plus or minus the distance (X′) of the point east or west from the central meridian (Y-axis); and the Y-coordinate equals the perpendicular distance to the point from the X-axis. The linear unit of the State coordinate systems is the foot of 12 inches defined by the equivalence: 1 international meter = 39.37 inches exactly.

The linear distance between two points on a State coordinate system, as obtained by computation or scaled from the grid, is termed the GRID LENGTH of the line connecting those points. The angle between a line on the grid and the Y-axis, reckoned clockwise from the south through 360°, is the GRID AZIMUTH of the line. The computations involved in obtaining a grid length and a grid azimuth from grid coordinates are performed by means of the formulas of plane trigonometry.

A property description by metes and bounds might include points located by coordinates as follows: "Commencing at U.S. Coast and Geodetic Survey Monument 'Bradley, Va', having coordinates y = 75,647.13 ft and x = 35,277.48 ft, as based on the Virginia Coordinate System, North Zone, as are all the coordinates, bearings, and distances in this description; thence S 36°30'E, 101.21 ft to the intersection of Able Street and Baker Avenue, whose coordinates are y = 75,565.77 ft and x = 35,337.45 ft, etc."

By Blocks, Tracts, or Subdivisions

In many counties and municipalities the land of the community is divided into subdivisions called BLOCKS, TRACTS, or SUBDIVISIONS. Each of these subdivisions is further subdivided into LOTS. Blocks and tracts usually have numbers, while a subdivision usually has a name. Each lot within a block, tract, or subdivision usually has a number.

From data obtained in a TAX MAP SURVEY or CADASTRAL SURVEY, a MAP BOOK is prepared which shows the location and boundaries of each major subdivision and of each of the lots it contains. The map book is filed in the county or city recorder's office, and henceforward, in deeds or other instruments, a particular lot is described as (for example): "Lot 73 of Tract 5417 as per map recorded in book 72, pages 16 and 17, of maps, in the office of the county/city recorder of (named) county/city"; or as "Lot 32 of Christopher Hills Subdivision as per, etc."

JOB REQUIREMENTS OF THE LAND SURVEYOR

In resurveying property boundaries and in carrying out surveys for the subdivision of land, the EA performing land surveys has the following duties, responsibilities, and liabilities:

1. Locate in the public records all deed descriptions and maps pertaining to the property and properly interpret the requirements contained therein.
2. Set and properly reference new monuments and replace obliterated monuments.
3. Be liable for damages caused by errors resulting from incompetent professional work.
4. Attempt to follow in the tracks of the original surveyor, relocating the old boundaries and not attempting to correct the original survey.
5. Prepare proper descriptions and maps of the property.
6. May be required to connect a property survey with control monuments so that the grid coordinates of the property corners can be computed.
7. Report all easements, encroachments, or discrepancies discovered during the course of the survey.
8. When original monuments cannot be recovered with certainty from the data contained in the deed description, seek additional evidence. Such evidence must be substantial in character and must not be merely personal opinion.
9. In the absence of conclusive evidence as to the location of a boundary, seek agreement between adjoining owners as to a mutually acceptable location. The surveyor has no judicial functions; he may serve as an arbiter in relocating the boundary according to prevailing circumstances and procedures set forth by local authority.
10. When a boundary dispute is carried to the courts, he may be called upon to appear as an expert witness.
11. He must respect the laws of trespass. The right to enter upon property in conducting public surveys is provided by law in most localities. In a few political subdivisions, recent laws make similar provision with respect to

private surveys. Generally, the military surveyor should request permission from the owner before entry on private property. When lacking permission from an adjoiner, it is usually possible to make the survey without trespassing on the adjoiner's land, but such a condition normally adds to the difficulty of the task. The surveyor is liable for actual damage to private property resulting from his operations.

A primary responsibility of a land surveyor is to prepare boundary data which may be submitted as evidence in a court of law in the event of a legal dispute over the location of a boundary. The techniques of land surveying do not vary in any essential respect from those used in any other type of horizontal-location surveying–you run a land-survey boundary traverse, for example, just as you do a traverse for any other purpose. What distinguishes land surveying from other types of surveying is the fact that a land surveyor is often required to decide the location of a boundary on the basis of conflicting evidence.

For example: suppose you are required to locate, on the ground, a boundary line which is described in a deed as running, from a described point of beginning marked by a described object, N 26°15′E, 216.52 ft. Suppose you locate the point of beginning, run a line therefrom the deed distance in the deed direction, and drive a hub at the end of the line. Then you notice that there is, a short distance away from the hub, a driven metal pipe which shows signs of having been in the ground a long time. Let's say that the bearing and distance of the pipe from the point of beginning are N 26° 14′E, 215.62 ft.

You can see that there is conflicting evidence here. By deed evidence the boundary runs N 26°15′E, 216.52 ft; but the evidence on the ground seems to indicate that it runs N 26°14′E, 215.62 ft. Did the surveyor who drove the pipe drive it in the wrong place, or did he drive the pipe in the right place and then measure the bearing and distance wrong? The land surveyor, on the basis of experience, judgment, and extensive research, must frequently decide questions of this kind. That is to say, he must possess the knowledge, experience, and judgment to select the best evidence when the existing situation is conflicting.

There are no specific rules which can be consistently followed. In the case mentioned, the decision as to the best evidence might be influenced by a number of considerations. The pipe is pretty close to the deed location of the end of the boundary. This might, everything else being equal, be a point in favor of considering the pipe bearing and distance, rather than the deed bearing and distance, to be correct. If the pipe were a considerable distance away, it might even be presumed that it was not originally intended to serve as a boundary marker. Additionally the land surveyor would consider the fact that, if the previous survey was a comparatively recent one done with modern equipment, it would be unlikely that the measured bearing to the pipe would be off by much more than a minute or the distance to the pipe off by much more than a tenth of a foot. However, if the previous survey was an ancient one, done perhaps with compass and chain, larger discrepancies than these would be probable.

Further considerations would have to be weighed as well. If the deed said, "From (point of beginning) along the line of Smith N 26°15′E, 216.52 ft", and you found the remains of an ancient fence on a line bearing N 26°15′E, these remains would tend to vouch for the accuracy of the deed bearing, regardless of a discrepancy in the actual bearing of the pipe or other marker found.

To sum up: in any case of conflicting evidence, you should (1) find out as much as you can about all the evidential circumstances and conditions, using all feasible means, including questioning of neighboring owners and local inhabitants and examination of deeds and other documents describing adjacent property, and (2) select the best evidence on the basis of all the circumstances and conditions.

As in many other professions, the surveyor may be held liable for incompetent services rendered. For example, if the surveyor has been given, in advance, the nature of the structure to be erected on a lot, he may be held liable for all damages or additional costs incurred as a result of an erroneous survey; and pleading in his defense that the survey is not guaranteed will not stand up in court. Since a civilian professional surveyor must be licensed before he can practice his profession, he must show that degree of prudence, judgment, and skill reasonably expected of a member of his profession.

LAND-SURVEY GENERAL PROCEDURE

As there are no universal rules for the weighing of evidence, so there are no universal, unvarying rules for land-survey procedure. The typical problem, however, usually breaks down into the following major action phases.

1. The location, study, and (when necessary) interpretation of all the available deeds, contracts, maps, wills, or other documents which contain a description of the boundaries. The principal repository for most of these instruments is usually the files in a city or county records office. The mere deciphering of ancient, handwritten documents is an art in itself. And here again it is not unusual to encounter conflicting evidence, in the shape of documents which purport to describe the same property, but which describe it differently. Or you may find a document in which some of the languages may bear more than one interpretation. In this last case you apply, as well as you can, a legal maxim which goes to the effect than an ambiguous document should be given the sense which the maker of the document may be reasonably presumed to have intended.
2. The determination, after study of all the documents and related evidence, of what the true property description may be presumed to be, and from this a determination of what physical evidence of the boundary locations exists in the field. Physical evidence means for the most part MONUMENTS. In land-surveying parlance, the term MONUMENT applies to any identifiable object which occupies a permanent location in the field and serves as a reference point or marker for a boundary. A monument may be a NATURAL monument, such as a rock, a tree, or the edge of a stream; or it may be an ARTIFICIAL monument, such as a pipe or a concrete monument. Do not use perishable markers for monuments, such as a wooden marker which decays easily.
3. The location, in the field, of the existing physical evidence of the boundaries.
4. The establishment of the boundary. This involves those decisions previously mentioned as to the best evidence. It also involves the setting, referencing, and marking of points which should have been marked in previous surveys but weren't, or which were marked with markers which have since disappeared.
5. The preparation of the property description.

PLATS OF SURVEYED LANDS

The official plat of a township or other subdivision is the drawing on which is shown the direction and length of each line surveyed, established, retraced, or resurveyed; the relationship to adjoining official surveys; the boundaries, designation, and area of each parcel of land; and, insofar as practicable, a delineation of the topography of the area and a representation of the culture and works of man within the survey limits. A subdivision of the public lands is not deemed to have been surveyed or identified until the notes of the field survey have been approved, a plat prepared, the survey accepted by the Director of the Bureau of Land Management as evidenced by a certification to that effect on the plat, and the plat has been filed in the district land office. Figure 6-26 shows a typical township plat. The original drawing shows both a graphical scale and a representative fraction for both the township as a whole and for the enlarged diagram. Because the plat has been photographically reduced, the representative fraction and scale are no longer true. Plats are drawn on sheets of uniform size 19" X 24" in trimmed dimensions, for convenience in filing. The usual scale is 1" = 40 chains, equivalent to a representative fraction of 1:31,680. Where detail drawings of a portion of the survey area are required, scales of 1 inch equals 20 chains or 1 inch equals 10 chains may be used. A detail of a small area may be shown (fig. 6-25), as an inset on the main plat. Larger details are drawn on separate sheets. When the drawing is simple, with few topographic or hydrographic features or works of man to be shown, the entire drawing is in black ink. When, as in figure 6-26, the features other than the survey lines are quite extensive, color printing is

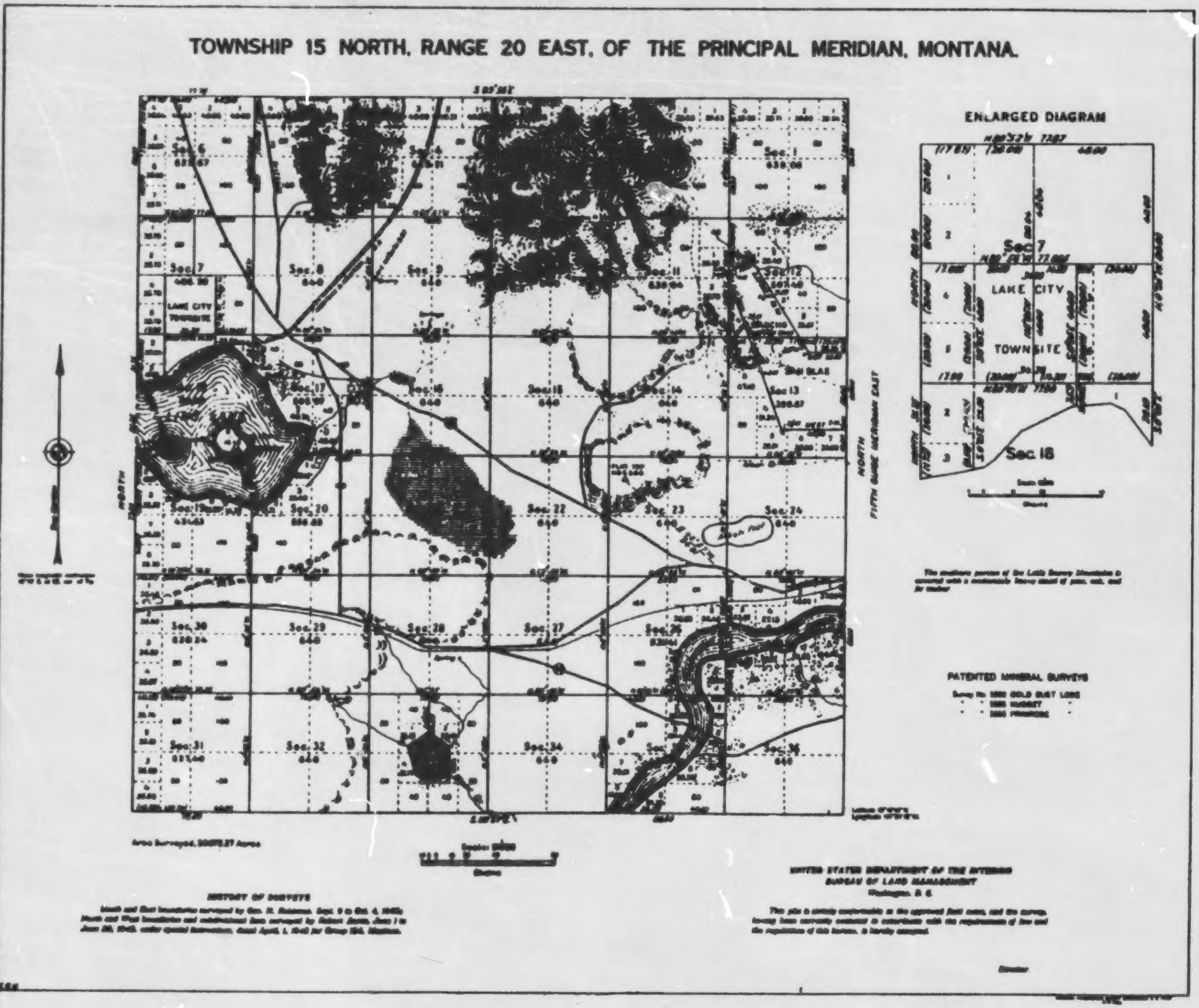

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Figure 6-28.—Typical township plat.

used. Survey lines, numbers, lettering, and railroads are printed in black; topographic relief, roads, highways, trails, culture, alkali flats, sandy-bottom draws, and sand dunes are shown in brown; rivers, lakes, streams, and marshes are shown by conventional symbols in blue; and timbered areas are indicated in green. Where such a green overprint might obscure other details, the presence of timber may be indicated in a note (fig. 6-26). These several colors are not shown on the reproduction of the plat presented in fig. 6-26, although the various features are indicated in appropriate colors on the original map where this figure was reproduced.

A property plat plan must contain the following:

1. Directional orientation, usually indicated by NORTH arrow.
2. Bearing and distance of each boundary.
3. Corner monuments.
4. Names of adjacent owners, inscribed in areas of their property shown.
5. Departing property lines. A departing property line is one which runs from a point on one of the boundaries of the surveyed lot through adjacent property. It constitutes a boundary between areas belonging to two adjacent owners.
6. Names of any natural monuments which appear on the plat (such as the name of a stream); or the character (such as "10-in. oak tree") of any natural monuments which have no names.
7. Title block, showing name of owner, location of property, name of surveyor, date of survey, scale of plat, and any other relevant data.

The preceding items are those which usually appear on any plat. Some land surveyors add some or all of the following as well.

1. Grid lines or "ticks" (a grid "tick" is a marginal segment of a grid line, the remainder of the line between the marginal ticks being omitted), when determinable.
2. On a plat on which grid lines or ticks are shown, corner locations by grid plane coordinates.
3. Streams, roads, wooded areas, and other natural features, whether or not they serve as natural monuments.
4. Surveyor's certificate. This is a statement (required by law in many states) in which the surveyor makes personal affidavit as to the accuracy of the survey. A typical certificate might read as follows: "I, (surveyor's name), registered land surveyor, hereby certify that this plat accurately shows property of (owner's name), as acquired in Deed Book 60, page 75, of the land record of (named) County, State of (name)."
5. The area of the property.

LAND SURVEY PRECISION

Most land surveying of tracts of ordinary size is done by transit-tape. For a large tract, however (such as a large Government reservation), corners might be located by triangulation—or primary horizontal control might be by triangulation and secondary control by supplementary traversing.

The precision used for land surveying varies directly with the value of the land, and also with such circumstances as whether or not important structures will be erected adjacent to the property lines. Obviously, a tract in lower Manhattan, New York (where land may sell for more than a million dollars per acre) would be surveyed with a considerably higher precision than would be used for surveying a rural tract.

Again there are no hard-and-fast rules. However, the prescribed order of precision for surveying the boundaries of a naval station might require the following:

1. Plumb bobs used for alignment and to transfer chained distances to the ground.
2. Tape leveled by Locke level.
3. Tension applied by spring balance.
4. Temperature correction.
5. Angles turned 4 times.

If you turn angles 4 times with a 1-minute transit, you are measuring angles to approximately the nearest 15 seconds. The equivalent precision for distance measurement would be measurement to the nearest 0.01 ft. Four-time angles might be precise enough for lines up to 500.00 ft long. For longer lines, a higher angular precision (obtained by repeating 6 or 8 times) might be advisable.

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CHAPTER 7

TOPOGRAPHIC SURVEYS

Topographic surveys are made to obtain field data from which topographic maps may be made, indicating the relief, or the configuration of the earth's surface, and the location of natural and man-made objects.

The objectives of topographic surveying include:

1. Establishing horizontal control.
2. Determining vertical control.
3. Determining horizontal location and elevation of a sufficient number of ground points to provide data for the map.
4. Locating such other natural or man-made details as required.
5. Calculating angles, distances, and elevations.
6. Plotting and finishing the topographic map.

To accomplish the forementioned objectives, various methods are employed to produce topographic maps. The location (both horizontal and vertical) of topographic details by transit stadia from traverse stations is described in *Engineering Aid 3 & 2*. In this chapter the general approach to the mapping problem, from the party chief's viewpoint, will be described with detailed reference to the planetable methods of locating details.

This chapter also contains a section devoted to surveys in support of geology and pedology, which are related to the use of topographic maps.

TOPOGRAPHIC SURVEYS

The procedures to be used in producing a topographic map depend on the use to which the map is to be put and the time and facilities available. Under some circumstances it is more economical to use aerial photogrammetry; in these cases the fieldwork is limited to establishing horizontal and vertical control, checking, and perhaps picking up some extra details. Some of the factors which affect the decision as to whether topo should be flown or shot in the field are the size of the site, the purpose of the map as reflected in the scale and contour interval needed, the denseness of the underbrush (which obscures the bottoms of swales and ravines), the types of trees and the time of the year as reflected in whether or not leaves are on the trees.

The methods to be used in a field topo survey depend largely on the purpose of the map. For example, the horizontal and vertical control may not have to be as precise, and the detail as extensive, for a 1" = 200' and 5' contour interval map to be used for preliminary planning as for a 1" = 50' and 2' contour interval map to be used for design of streets, utilities, and site grading.

DEVELOPMENT OF TOPOGRAPHIC MAPS

Typical steps in the development of a map at the latter scale might be as follows: First, gather all available maps, plats, survey data, and utilities data which pertain to the site and study them carefully. Consider the boundaries of the site in relation to the intended use of the topo map. If the map is to be used for design purposes, certain off-site information will be even more important than on-site details. For example, the location and elevations of utilities and nearby streets is vital. The location of drainage divides above the site and details of outfall swales and ditches below the site are necessary for the design of the storm drainage facilities. Topographic details of an off-site strip of land all around the proposed limits of construction are necessary so that grading can be designed to blend with adjacent areas. Decide on the datum and bench marks to be used; consider previous local surveys, USC&GS monuments, sanitary sewer inverts (not rims—they are frequently adjusted) and assumed datum. Determine whether or not there is a coordinate system in the area monumented sufficiently for your use; if not, plan on using assumed coordinates. In the latter case, decide on the source of the meridian: adjacent surveys, magnetic, assumed, or shooting the sun or Polaris.

Next, perform a reconnaissance survey. Observe the vegetation and decide how many men you will need to cut brush. Select main control traverse stations at points appropriate for plane-table setups. Decide on the number and location of cross ties or secondary traverse lines needed to provide sufficient plane-table stations. Select these points so that plane-table setups will have to be extended only a minimum distance before checking back into control.

The next step is to run the traverse lines, checking its directions from time to time where necessary on long traverse. Checks could be done by astronomical methods, by cut-off lines, or by connecting the traverse with established points. Then run the levels, turning on all traverse stations. Close, balance, and coordinate the main traverse. Then adjust the cross ties into the main traverse. Balance the levels. Plot the traverse stations by coordinates on the plane-table sheets (milar or stabilene sheets are ideal). Be sure there is sufficient overlap of all sheets. Be sure there is sufficient control on each sheet for orientation, and for extension of setups (if necessary). Number the traverse stations with the same numbers marked on the guard stakes in the field and show the elevations.

The planetable work is the final big step of the fieldwork. But some transit and level work may still need to be done. The location of some details (such as street centerlines or buildings) may be needed to a precision greater than that obtainable with the planetable; tie such details to the traverse by transit tape survey. For design purposes, the elevation of some points (such as the inverts of culverts, paved flumes, and sewers, and the tops of curbs and gutters) may be needed to a precision greater than that obtainable with the planetable. Use the level to obtain such elevations. The final step in the production of the topographic map is, of course, tracing the information from the planetable sheets onto the final drawing.

Random traversing, as described in the foregoing, is not the only way of establishing horizontal control. Grids are frequently used. One good way of identifying grid lines is to assign a letter to each line in one set, and then run stationing along each line. Another method is used in the example described in the following paragraphs.

Suppose that a site chosen through reconnaissance for an advanced base with airstrip facilities is as shown in figure 7-1. Here there is a sheltered water area for a potential harbor, a strip of woodland extending back from the shore, and then a strip of clear level country where an airstrip could be constructed.

Topographic data for a map of this area might be gathered by three field parties, two of them transit-level parties and the third a planetable party. The transit-level parties would operate in the wooded and the water areas, the planetable party in the clear area.

Basic horizontal control is the MAIN BASE LINE, run along the edge of the wooded area as shown. Topographic details in the clearing will be plotted from planetable stations tied to the

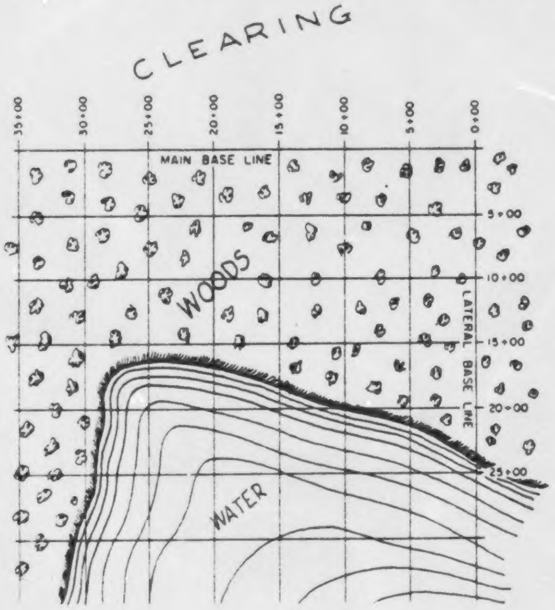

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Figure 7-1.—Advanced base site.

main base line. Details in the wooded area and offshore will be plotted from stations on a grid tied to the main base line.

Transit-level party #1 runs the main base line from station 0 + 00, located at random; setting hubs at every 500-ft station. Transit-level party #2 runs a LATERAL base line from 0 + 00, perpendicular to the main base line, and sets hubs at every 500-ft station. From every 500-ft station on the main base line, party #1 will run a LATERAL, perpendicular to the main base line (and therefore parallel to the lateral base line, party #2 will run a LONGITUDINAL, perpendicular to the lateral base line (and therefore parallel to the main base line).

You can see that the coordinates of a point of intersection between a longitudinal and a lateral are the designations of the longitudinal and the lateral-similarly. The coordinates of any point in the grid area are the main base line station and the lateral base line station of lines perpendicular to the main base line and the lateral base line passing through the point. You designate any point by its grid coordinates, expressed in fractional form, one over the other. You must decide which coordinate you will place on top, and then BE SURE TO STICK TO YOUR RULE. We'll place the lateral coordinate (that is, the main base line station) on top. For the point of intersection between lateral 15 + 00 and longitudinal 10 + 00, for example, our designation will be 1500/1000.

With regard to the vertical control situation, it may be the case that there are no established bench marks in the area. If this is so, the level group from party #2 should take a series of rod readings, over a succession of high and low tides, or on the high-water mark wash line along the beach. The average of all these readings may be used as a temporary vertical control datum, until a more accurate datum is obtained from tide gage readings. From a temporary BM at or near the beach, a line of levels can be run to station 0 + 00 on the main base line. Temporary elevations of hubs in the main base line and the lateral base line can then be determined.

Finally, the transit-level parties will shoot the detail in the vicinity of each of the 500-ft points of the intersection on the grid.

DETAIL BY PLANETABLE

The planetable party will be engaged in the process of locating detail and drafting a map of the clear area in a single operation.

A planetable field party for a large survey should consist of an instrumentman or topographer, a notekeeper or computer, and one or more rodmen. The instrumentman operates the planetable and alidade, making the observations and performing the plotting and sketching. He reads off the rod readings and vertical angles to the notekeeper, who records and reduces the field notes. The notekeeper computes the elevations and the horizontal distances. The rodman's job is to occupy the minimum number of points required to give an adequate representation of the ground being surveyed.

Planetable Equipment

For regular topographic mapping, a 24" X 30" planetable is generally employed. An alidade and stadia rod or Philadelphia rod are used in combination with the planetable. With these instruments, the direction, the distance, and the difference in elevation can be measured, computed, and plotted directly in the field. The planetable operation produces a completed sketch or map without need for further plotting or computing. Mistakes are easily recognized and corrected right in the field.

A small table called a traverse table, about 18" X 24", is often used for reconnaissance sketching and small-scale mapping. Some traverse tables are equipped with a ruler sight alidade with hinged sights similar to those on a surveyor's compass. Others merely contain a scale, the edge of which is used for sighting. A trough compass is countersunk along one edge of most traverse tables to facilitate orientation.

Special weather-resistant drawing paper is available for planetable work. The paper should be attached before the board is oriented and leveled.

Planetable Methods

There are four common methods of orienting the planetable; they are radiation, progression, intersection, and resection. Each of these methods is discussed separately below.

RADIATION.–In this method the detail in a circular area around the planetable is plotted from a single setup. Select a point on the drawing paper to represent the point on the ground. The planetable is set up over the point on the ground whose position has been previously plotted, or will be plotted, on the planetable sheet during the operation. The board is oriented either by using a magnetic compass for north-south orientation, or by sighting on another visible point whose position is plotted. The board is clamped and the alidade is pointed toward any new desired point using the plotted position of the setup ground station as a pivot. A line drawn along the straightedge, which is parallel to the line of sight, will give the plotted direction from the setup point to the desired point. Once the distance between the points is determined, it is plotted along the line to the specified scale. The plotted position represents the new point at the correct distance and direction from the original point. By holding the planetable orientation and pivoting the alidade around the setup point, the direction to any number of visible points can be quickly drawn. The distances to these points, determined by any convenient method as prescribed by the desired accuracy, can be plotted along their respective rays from the setup point. Thus, from one setup, the positions of a whole series of points can be established quickly. For mapping, the difference in elevation is also determined and plotted for each point. The map is completed by subdividing the distances between points with the correct number of contours spaced to represent the slope of the ground.

In clear, level country, detail within a radius of about 1500 ft can be located with reasonable accuracy. This means that, from four setups, an area of about a square mile can be covered. The clear area shown in figure 7-1 measures 3500 by about 2000 ft, or just about one-third square mile. Figure 7-2 shows how this rectangular clear area could be covered, with considerable overlap to spare, from two instrument points tied to the midpoint of the main base line.

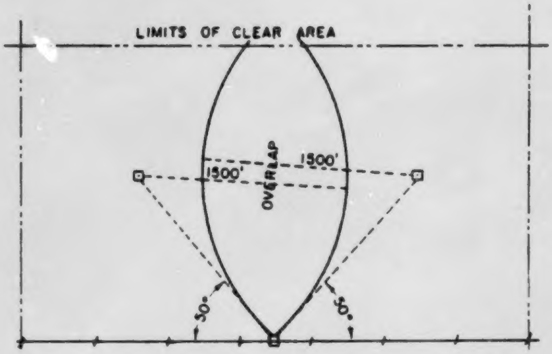

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Figure 7-2.–Planned planetable stations to cover a rectangular area.

PROGRESSION.–In radiation, as you can see, successive planetable instrument points are located by triangulation. In progression, the planetable might be said to generate a traverse as it moves along. Figure 7-3 illustrates the method. Here the planetable progressed from station A through B, C, and D to E, thus plotting the closed traverse ABCDE. You locate your starting point on the paper so as to ensure that all the other stations on the traverse will lie within the margins of the paper, which of course involves selecting an appropriate distance scale as well.

Set up the table so that starting point a on the paper is directly over station A on the ground. Orient the board by aligning the edge of the blade with point a, sighting through the telescope on station B, and then pivoting the board so as to bring line ab (to be drawn along the edge of the blade) where you want it to come on the paper.

Determine the horizontal distance from station A to station B, and lay off ab to scale. Then shift to station B, plumb b on the paper over B on the ground, set the edge of the blade on b, backsight on station A, and bring ba in line with the edge of the blade. Then proceed with station C as you previously did with B.

INTERSECTION.–When two points which can be occupied by the planetable have already been plotted on the paper, the location of a third point can be plotted by determining the point of intersection of lines of direction from the already plotted points. This method, known as intersection, is illustrated in figure 7-4.

You wish to locate point X, and you have A and B plotted. Plumb point A on the paper over

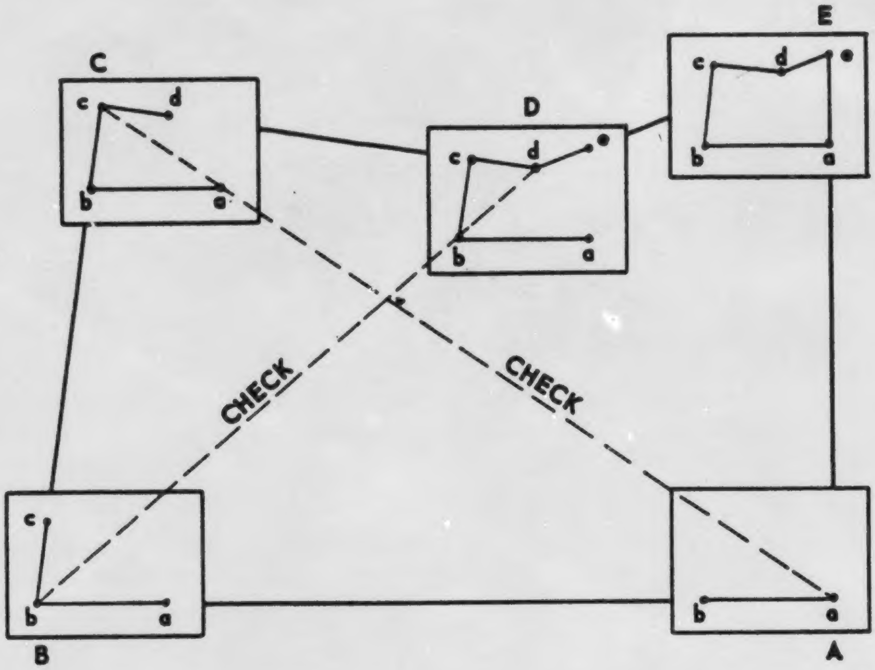

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Figure 7-3.—Progression.

station A on the ground, set the edge of the blade on A, and line up the edge with line AB, as indicated in figure 7-4. Then revolve the board until the telescope is trained on point B on the ground. Now, keeping the edge of the blade on A on the paper, train the telescope on point X on the ground. The edge of the blade is now on the line from A on the ground to X on the ground; draw a line along the edge from A toward X.

Now shift the planetable to B on the ground, and repeat the procedure you carried out at A. You will wind up with two lines on the paper, one from A, the other from B, toward X. The point where these two lines intersect is the plotted location of X.

RESECTION.—This method, like intersection, is one in which you have two points plotted and desire to locate a third. It varies from intersection in that, instead of occupying the already plotted points with the planetable, you occupy instead the point whose location is being sought.

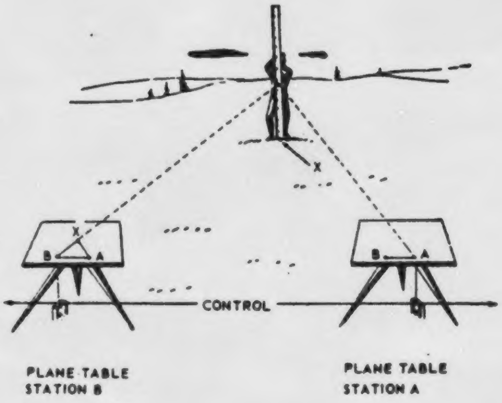

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Figure 7-4.—Intersection.

Figure 7-5 illustrates the method. This figure shows one-point resection. Here you have a point of known location, A, and a point, X, whose location is desired. First measure the

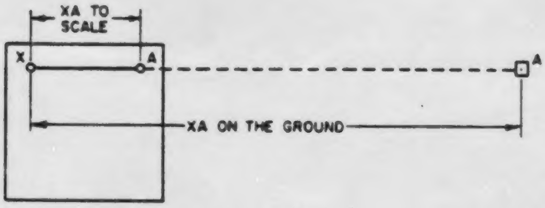

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Figure 7-5.—One point resection.

horizontal distance between A and X by an appropriate method. Then set up the planetable at X, train the telescope on A, orient the board so that XA will lie where you want it on the paper, and draw a line along the blade from A toward X. Lay off AX to scale on this line.

In THREE-POINT resection (familiarly called the THREE-POINT problem) you determine the location of a point with reference to three points of known location. The method is frequently used to locate minor triangulation stations with reference to major stations. Two common solutions are the LEHMANN TRIANGLE OF ERROR solution and the TRACING CLOTH or MECHANICAL solution.

Figure 7-6 illustrates the Lehmann solution. The figure shows three located points: A, B, and C. The planetable is set up over D, a point whose location is desired, and oriented as closely as possible, either by compass or by eye.

If the table were oriented correctly, resection lines from A, B, and C would intersect only at a point, d. In most cases, however, these lines intersect to form a small triangle (a′b′c′ in fig. 7-6), called the TRIANGLE OF ERROR. The correct location of d is at the center of this triangle.

In figure 7-6 the planetable is set up over a point which is inside the triangle formed by stations A, B, and C. Therefore, the plotted position d lies at the center of the triangle of error, and it is fairly easy to estimate where this center is. However, it could be the case that the point whose location is sought may lie outside of the triangle formed by the three located points. In a case of this kind you would use the tracing-cloth solution.

Observe figure 7-7, which illustrates the tracing-cloth solution. Here there are three located points, A, B, and C, and a point, P, whose location is desired, lying outside the triangle formed by A, B, and C. First, set up the planetable, on which is mounted the paper with a, b, and c plotted thereon, over P, and orient it as closely as possible. Then fasten a sheet of tracing cloth or transparent paper over the board, and locate P′ by sighting on A, B, and C. Draw in P′a′, P′b′, and P′c′.

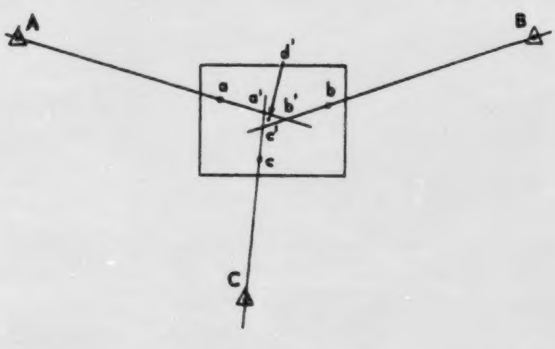

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Figure 7-6.—Triangle of error.

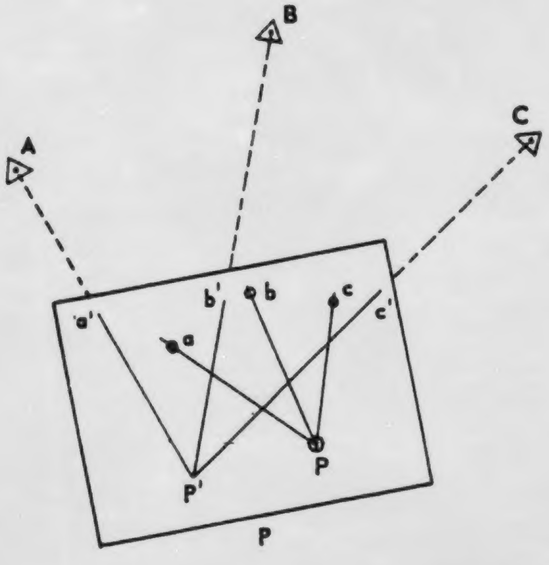

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Figure 7-7.—Tracing cloth solution.

Now unfasten the tracing cloth, and move it into a position where these three lines pass through plotted points a, b, and c. P' on the tracing cloth will now be located at the correct location of point P.

Values of Planetable Method

Advantages of the planetable method of topographic surveying are as follows:

1. The map is made directly in the field, thus combining the data-collection and drafting into a single operation. The area under survey is visible as a whole, which tends to minimize the overlooking of important data. Errors in measurement may be easily checked by taking check observations on a prominent point whose position has been plotted on the map. If the edge of the blade does not contact the proper point or points, an error is indicated. An error thus located can be easily corrected on the spot.
2. The planetable method greatly reduces the number of field notes required, and consequently the number of computations. This in turn reduces the number of opportunities for errors and mistakes.
3. The graphic solutions of the planetable are much quicker than the same solutions by methods requiring angular measurements, linear measurements, and computations. Thus a great deal more area can be covered in much less time.
4. When the country is open and level, the planetable topographer has a wider choice in the selection of detail points. He need not be hampered by backsight-foresight requirements. He can locate inaccessible points easily by graphic triangulation, or quickly determine the location of a point with reference to one, two, or three points of known location.

Disadvantages of the planetable method are as follows:

1. The planetable and its plotting and drawing accessories are more difficult to transport than transit-stadia equipment.
2. Weather not bad enough to rule out transit-stadia will make planetable work impossible.
3. The use of the planetable is limited to relatively level, open country.

SURVEY SUPPORT FOR GEOLOGY AND PEDOLOGY

This section discusses surveys in support of geology and pedology. In essence, it is a topographic survey, however, you must be aware of the other specialized data that may be included as required by the Geologist or the Soil Engineer when collecting data for engineering studies for naval construction projects.

SURVEY SUPPORT FOR GEOLOGY

The end product of most topographic surveys is a topographic map. In geology or other related sciences, the topographic survey is the first part of a series of interrelated surveys, the end product is a map containing not only topographic information, but also other specialized data keyed to it. In geologic surveys, a geologist makes systematic observations of the physical characteristics, distribution, geologic age, and structure of the rocks as well as the ground water and mineral resources that the rock contain. These observations are expressed in finished form as geologic maps and texts. The objective of the geological survey is to portray, in plan or in cross section, geological data required for subsequent constructions or for other uses.

Pure geologic data has little direct application to naval problems, however, if the field information is interpreted into specialized lines, it is of considerable use in Naval Construction Forces planning and operations. Construction Forces requirements may necessitate regional geologic study and mapping, surveys of more limited areas, or the development of detailed geologic data at a construction site.

Methods of Geologic Surveying

Most geologic data is gathered from an examination of rocks in the field. In addition, examination of drainage and relief patterns on detailed maps or aerial photographs provides considerable supplementary data on rock structures and distribution.

In the field, the geologist conducts his survey by examining the rock, whether it is exposed at the surface and not covered by soil or other

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material. At such exposures, called OUTCROPS, he systematically records the physical characteristics of the rock, thickness of exposure, inclination of rock bedding, and development of joints or fractures. In addition, the age of the rock is determined from fossils or the sequence of rock units. Rock investigations are not confined to surface exposures, as the deeper seated rocks are examined by using samples obtained from auger or boreholes. The information gathered by the geologist is placed on a map base by plotting the rock types in color with other data incorporated as symbols or annotations. To amplify the map data, more complete descriptions of outcrops are entered in notebooks with the entries keyed to the field map. Surveyors support the geologist by preparing basic topographic maps on which the results of geologic investigations are plotted and by making such tie measurements to geologic features as the geologist may require.

The geologist uses simple survey methods in plotting geologic features on a field map. Where an outcrop can be located with reference to a cultural or relief feature, it is generally plotted on a map by spot recognition. In other cases, the relation of a geologic feature to a recognizable topographic feature is established by using a magnetic compass to determine direction, and by pacing or taping to measure distance. Slope or small differences in elevation are measured by using a clinometer or hand level, while an altimeter is used where there are large differences in elevation. When the geologic survey is keyed to a large-scale plan, the geologist generally uses a planetable and data is plotted with accuracy commensurate with the accuracy of the base plan.

Base Map Surveys

The survey for the base map should precede the geologic survey, because the geologist uses the map in the field to plot his data and to determine his position by identification of topographic details. If aerial photographs are available, the base map need not be made before the geologic survey, since the geologist can use the aerial photograph as a plotting base and later transfer the data to a base map. However, where possible, the base map should be prepared in advance as the number of aerial photographs needed to cover an area is generally too large to be handled in the field.

Planetable topography is the method best suited to relatively open country. In the absence of detailed instructions, the following specifications are generally satisfactory:

1. BASE DIRECTION. To determine a base direction, take from a known base, a side in a triangulation net, or a course of a basic control traverse.
2. LOCAL HORIZONTAL CONTROL. Use planetable traverses run in closed circuits or between known control stations of a higher order of accuracy, or locate planetable stations by graphical triangulation.
3. LOCAL VERTICAL CONTROL. Where the terrain is relatively level, carry elevation along traverses by vertical angle or stadia-arc measurements, adjusting elevations on closure at a basic control station. For rugged terrain mapped at one of the larger contour intervals, barometric or trigonometric leveling is suitable.
4. SIGHTS. Use telescopic alidade.
5. DISTANCE MEASUREMENTS. Use, in general, stadia or graphical triangulation to locate points and station. Certain measurements can be made most conveniently by pacing or rough taping.
6. CONTOURING. Locate and determine the elevations of controlling points on summits, in valleys and saddles, and at points of marked change of slope. Interpolate and sketch contours in the field, using these elevations for control.
7. ACCURACY. Distance measurements by stadia should be accurate to 1 part in 500. Sideshot points located by pacing or other rough measurements should be accurate to within 25 ft. Sights for traverse lines or graphical triangulation should be taken with care to obtain the maximum accuracy inherent in the telescopic alidade. The error in the elevation of any point, as read from the finished map, should not exceed one-half of the contour interval.

Topography may be located more conveniently in heavily timbered country by stadia measurements from transit-stadia traverse than by the use of the planetable, although the time required for plotting will be increased. The specifications listed above are generally applicable. Read horizontal angles on traverses to 1 minute, and horizontal angles for side shots which will be plotted by protractor to the nearest quarter-degree. Read vertical angles for elevation determination to 1 minute or use the stadia arc. Keep complete and carefully prepared stadia notes and sketches to assure correct plotting.

When the geologist indicates that a map of a lower order of accuracy will fulfill his needs, planetable or compass traverses are suitable.

Use of Aerial Photographs

If aerial photographs are available, the geologist generally uses them in the field in lieu of a map. The most satisfactory results are obtained from large-scale photographs 1:15,000 or larger. Some topographic features, such as some ravines, rocky knobs, or sinkholes, are too small to be shown on maps. These features, as well as the larger topographic forms such as stream channels and swamps, can be observed directly from aerial photographs. The photos also can be used to prepare a base map for portrayal of the field data by tracing planimetric detail from an uncontrolled mosaic with spot elevations added from field surveys. Use of contact prints of aerial photographs by the geologist in place of the base map is satisfactory, except where large scale plans for engineering purposes are to be the base. In such a case the distortion within an aerial photograph does not permit plotting of geologic data commensurate with the accuracy of the final plan.

Map Bases for Detailed Geologic Surveys

Detailed geologic surveys generally cover a specific map area, geographic region, or specified site from scales of 1:62,500 to 1:600 or larger. In general, the very large scales are used for specific engineering or mineral development problems.

SITE PLANS AND PROFILES.–Geologic data affecting foundation design at construction sites are plotted on plans drawn to scales of 1 inch = 50, 100, 200, or 400 feet. Contour intervals may range from 1 to 10 feet, depending upon the roughness of the terrain. Planetable mapping is suited to plotting the topographic features, ranges, and reference points used to locate drill holes, rock outcrops, and other geologic data. When plotting contours on a 1- or 2-foot interval it is better to locate points which are actually on the contours or to determine elevations at the intersection of closely spaced grid lines staked out on the site rather than to use the method of contouring specified earlier in this section. In addition to a plan, the geologist may require that profiles be drawn along selected lines or that the boring logs of test holes be plotted to suitable scales.

USING A TOPOGRAPHIC MAP AS A BASE MAP.–The base map for a detailed geologic survey is a complete topographic map or plan with relief expressed by contours. Colors and symbolization of basic details are simple so that they will not conflict with the overlay of geologic information that is shown by colors and symbols. Published topographic maps are used where suitable. The geologic survey is expedited if the map base is from a quarter to double the scale of the map on which the information is to be presented. Enlargements of the base map are generally used to satisfy this requirement, rather than using other maps of a larger scale. This permits the direct reduction of geologic data to the scale of the final map with a minimum amount of drafting.

When no topographic map is available or if existing maps are not suitable, a base map or plan must be prepared from detailed topographic surveys. Culture and relief (contours) should be shown in the greatest detail possible. The survey for the base should conform to third order accuracy where large geographic areas are concerned. Maps made from aerial photographs using precise instrument methods, such as multiplex, can be used in place of field surveys. Altitude or elevation of the intersection of boreholes and the surface should be accurate to the nearest half-foot.

SURVEY IN SUPPORT FOR PEDOLOGY

If there is a requirement for pedological mapping for the purpose of locating the limits of sand or gravel deposits suitable for concrete aggregates, road materials, or for other construction operations, the pedological survey is conducted under the direction of the soils engineer, and the surveyors mission is one of support to the soils engineer's objective.

The engineer's objective in a pedological survey is to prepare data in plan and profile symbolizing soils and outcroppings on maps, overlays, and sketches for subsequent engineering uses. The following approaches may be used in conjunction with soils survey operation.

1. Aerial photography may be used when an extensive area is to be surveyed. Usually there are no survey measurements required in this case.
2. Maps of an area of several square miles in extent are required when an initial study or technical reconnaissance is needed for an engineering project. Low-order survey measurements usually suffice for the preparation of a reconnaissance sketch upon which the soils engineer can plot the pertinent data.
3. A sketch of an airfield, for example, is frequently required by the soils analysts before construction planning can be initiated. In this case the surveyor applies low-order measurements to prepare a sketch (1 inch = 100, 200, or 400 feet) upon which the soils engineer plots the results of soil tests and findings.

Aerial Photography

Photo coverage of the area under consideration facilitates the establishment of control for the pedological survey. The use of vertical aerial photographs in the planning phase of outlining ground control will speed the survey regardless of the size of the area to be covered. If controlled photographs are available, the survey engineer can locate points by pricking or keying them to the photographs. An uncontrolled photograph may be satisfactory for the surveys of low-order accuracy mentioned in the preceding paragraph. The survey party chief prepares, according to the soils analyst's instructions, maps or overlays upon which are plotted the control and ties to pedological features. The pedological interpretation of aerial photographs is the responsibility of the terrain analysts.

Planetable Traverse

The planetable traverse is best adapted to relatively open country for the preparation of the basic sketch upon which the soils engineer plots pertinent data. In the absence of detailed instructions from the soils engineer, the following procedures are generally satisfactory for preparing a sketch of an area of several square miles (3 miles by 3 miles maximum for initial exploration):

1. SCALE. 1:12,500 or 1:25,000.
2. TRAVERSE CONTROL. Run in circuits or between known positions of a higher order of accuracy.
3. SIGHTING. Use a peep-sight or a telescopic alidade.
4. DISTANCE MEASUREMENTS. Pace or rough tape. When a telescopic alidade is available, use stadia measurements where possible with a view to reducing the time required for the survey rather than increasing the accuracy.
5. BASE DIRECTION. To determine a base direction, select known bases: railroad or highway tangents, recognizable features, or reliable topographic maps. In the absence of these known bases, then use magnetic north as determined by compass observations.
6. COMPASS. Use military compass, forestry compass, or pocket transit.
7. DISTANCE BETWEEN BASIC CONTROL POINTS. Maintain 3 miles as the extreme maximum distance between stations.
8. ACCURACY. Distances should be measured in such a manner that points can be plotted within 25 feet. For the scales suggested, measurements to 1 part in 100 will suffice. Take sights with peep-sight alidade with care to maintain directions of an accuracy comparable to distances.
9. TOPOGRAPHY. Usually not required on reconnaissance surveys for pedology, particularly in areas of low relief. Where suitable deposits of sand, gravel, or stone have been located, route surveys from the site to the point of use may be required for the location of haulage roads, conveyors, or other means of transporting the material. In hilly terrain, rough topography, obtained by clinometer, pocket transit, or sta-

4/35

dia, may be required to facilitate the location of a favorable route.

Compass Traverse

The compass traverse is more convenient in heavily wooded areas although more time is required for plotting than is the case with planetable traversing. Traverse lines between stations should be long in order to reduce the number of observed bearings. Points between stations are located by offsets from the traverse lines. Where local attraction affects compass readings, points are plotted by intersection. Survey readings may be plotted in the field. Notes should be kept in case it is necessary to retrace the traverse. In the absence of detailed instructions from the soils engineer, the basic guides for planetable traverse apply.

Field Sheets and Site Plans

The survey engineer must furnish the soils analysts with suitable maps, overlays, and sketches for the plotting of pedological data. After the preparation of a reconnaissance field sheet of an area of several square miles, the soils analysts may require a sketch of a particular site in which many samples are taken for a more detailed study. In the absence of detailed instructions, the surveyor prepares a sketch on a scale of 1 inch = 400 feet and provides ranges and reference points to aid in plotting or tying in specific positions of auger holes, drill holes, and lines of exposed rock or other pedological features. For plotting the data of a range, cross section, or series of boreholes, the soils analyst may require the surveyor to provide a basic plot on a scale of 1 inch = 100 feet or of 1 inch = 200 feet. Survey measurements will be conducted accordingly.

CHAPTER 8

HORIZONTAL AND VERTICAL CURVES

The surveyed centerline of a road or highway consists of a series of straight lines and curves. Many people may rate a highway as smooth or bumpy; however, surveyors and engineers will be concerned about its physical and safety features. When you consider its horizontal alignment or changes in horizontal direction, you will be concerned with HORIZONTAL CURVES; when you think about slopes (the rise and fall), you will be concerned with VERTICAL CURVES. It is the introduction of these curves that makes modern travel more comfortable and enjoyable.

As an EA1 or EAC you might have to design these curves yourself; generally, however, your main concern is to compute for the missing curve elements and parts as problems occur in the field in the actual curve layout. You will find that a thorough knowledge of the properties and behavior of horizontal and vertical curves as employed in highway work will eliminate delays and unnecessary labor. Careful study of this chapter will alert you to common problems in horizontal and vertical curve layouts.

HORIZONTAL CURVES

When a highway changes horizontal direction, it is not feasible to make the point where it changes direction a point of intersection between two straight lines. The change in direction would be too abrupt for the safety of modern, highspeed vehicles. It is therefore necessary to interpose a CURVE between the straight lines. The straight lines of a road are called TANGENTS because the lines are tangent to the curves used to change direction.

In practically all modern highways, the curves are CIRCULAR curves; that is, curves which form circular arcs. The smaller the radius of a circular curve, the sharper the curve. For modern, high-speed highways the curves must be very flat, rather than sharp; meaning that they must be large-radius curves.

COMPUTATION OF HORIZONTAL CURVES

In highway work, the curves needed for the location of improvement of small secondary roads may be worked out in the field. Usually, however, the horizontal curves are computed after the route has been selected, the field surveys have been done, and the survey base line and necessary topographic features have been plotted. In urban work, the curves of streets are designed as an integral part of the preliminary and final layouts which are usually done on a topographic map. In highway work, the road itself is the end result and purpose of the design; but in urban work the streets and their curves are of secondary importance, and the best utilization of the building sites is of primary importance.

The design of the curve consists principally of selecting the length of the radius (or "degree of curvature," explained later). This selection is based on such considerations as the design speed of the highway and the sight distance as limited by headlights or obstructions (see fig. 8-1). Typical radii which you may encounter are 12,000 feet or longer on an interstate highway, 1,000 feet on a major thoroughfare in a city, 500 feet on an industrial access road, and 150 feet on a minor residential street.

ELEMENTS OF A CURVE

Refer to figure 8-2, which shows some of the elements of a circular curve.

P.C. Point of curvature. Also designated B.C. (beginning of curve) or T.C. (tangent to curve).

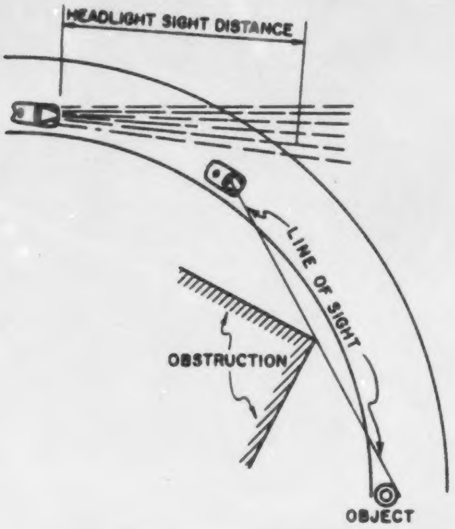

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Figure 8-1.—Lines of sight.

P.I.	Point of intersection of tangents. Also designated designated V (vertex).
P.T.	Point of tangency. Also designated E.C. (end of curve) or C.T. (curve to tangent).
Δ (I)	(Greek letter delta). Central angle; that is, the angle which subtends the total arc of the curve. Is always equal to the deflection angle at the P.I. Also designated I (intersection).
R	Length of the radius. The radius is always perpendicular to the back tangent at the P.C. and to the tangent ahead at the P.T.
L	Length of the curve. Also designated A (arc).
T	Tangent distance. The distance from the P.I. to the P.C., or the distance from the P.I. to the P.T. Said distances are always equal.
P.O.C.	Any point on the curve.

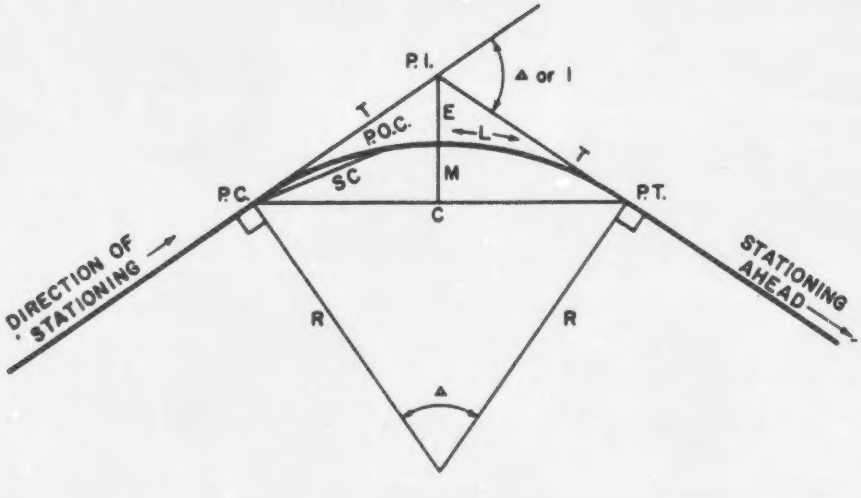

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Figure 8-2.—Curve elements.

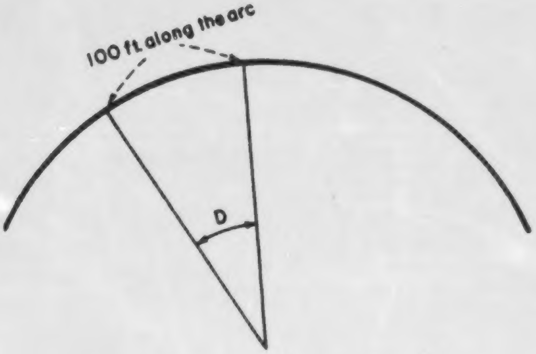
C Chord distance. The straight line distance from the P.C. to the P.T. Called the LONG CHORD (L.C.) to distinguish it from any other shorter chord, such as the one labeled SC in the illustration.

E External distance; from the P.I. to the midpoint of the curve. Also called the external secant.

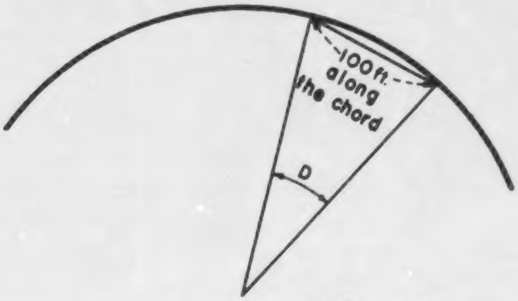
M Middle ordinate. Distance from the midpoint of the curve to the midpoint of the long chord.

DEGREE OF CURVATURE

An element of a curve that deserves separate attention is the DEGREE OF CURVATURE (D). Curvature may be identified by simply stating the length of the radius of the curve; this was done earlier in the chapter when typical radii for various roads were cited and is commonly done in land surveying, and in the design of urban streets. But in highway and railroad work, the curvature is defined by the degree of curvature. There are two definitions of the degree of curvature; both are illustrated in figure 8-3. According to the ARC DEFINITION, the degree of curvature is the central angle which subtends 100 feet measured along the ARC of the curve. According to the CHORD DEFINITION, the degree of curvature is the central angle which subtends a portion of the curve which has a CHORD of 100 feet. Therefore, if you take a sharp curve, mark off a portion so that the distance along the ARC is exactly 100 feet, and determine that the central angle for that portion is 12°, then you have a curve for which the degree of curvature (arc definition) is 12° or, as commonly stated, you have a "12° curve". If you take a flat curve, mark a 100-foot CHORD, and determine the central angle to be 0°30′, then you have a "thirty-minute curve, chord definition." The chord definition is used in railraod practice and in some highway work. The chord definition was adopted because of its practical application in the field. When you measure around a curve with a 100-foot tape, you are measuring a series of 100-foot chords. The arc definition is widely used in highway practice because it facilitates the complex computations needed for modern highway design.



ARC DEFINITION



CHORD DEFINITION

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Figure 8-3.—Degree of curvature.

CURVE FORMULAS

The relationship between the elements of a curve are expressed in a variety of formulas. Generally, if you know two elements, you can compute the other elements. This is not true, however, in the case of R and D, because as explained before, they are both expressions of curvature. Which factors are given will vary with the job. On a highway project, the Δ at the P.I. (or I) will have been measured in the field and then the design criteria will indicate the required D (or R). On urban streets, the Δ might be protracted from the preliminary plan and the T or R measured on the plan. The T and L must be computed in most cases so that you may properly station the curve (explained later.) The M or E may control the design of a curve or provide a check on the design of a curve to meet field conditions. Suppose that the alignment of a road is being established in the field, and that the location of the tangents has been decided upon (fig. 8-4). Therefore the Δ may be measured in the field, providing one given element for the computations. Suppose it is decided that the curve should be distance "a" off the center front of building A. Turn angle α (alpha), which is $\frac{180^\circ - \Delta}{2}$, at the P.I. and measure down to point A; the measured distance is the E of the curve. Thus a second element is provided, permitting the complete computation of the curve. Or suppose, in another case as shown in figure 8-5, that the alignment of the tangents has been determined, Δ measured, and a tentative P.C., P.T., and T selected. Suppose further that it is desired to check whether the resulting curve clears building B. Using the formulas presented in this section, compute C and M. Lay off 1/2 C, occupy point C, turn 90° and lay off M, setting point B. Then observe whether or not the clearance of building B is satisfactory. Of the many curve formulas that may be developed, the formulas in this section are a few that are commonly used in practice.

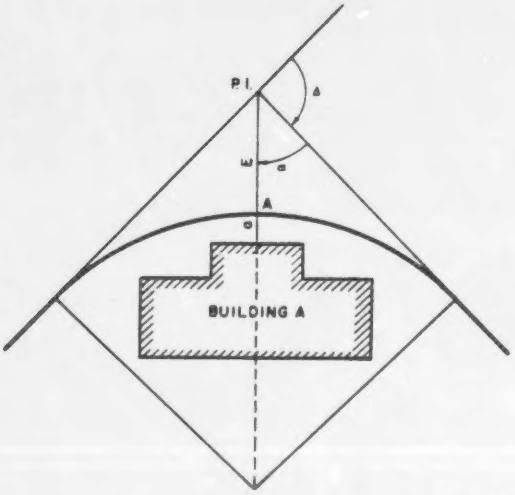

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Figure 8-4.—Measuring E.

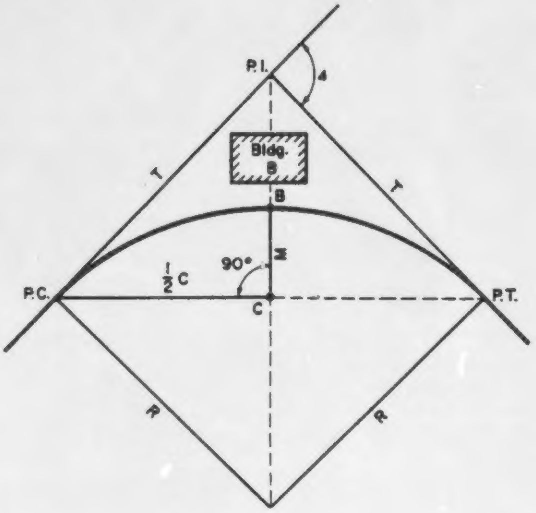

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Figure 8-5.—Checking by use of M.

Radius and Degree of Curvature

In the case of the arc definition, figure 8-6, the ratio between D (the angle subtended by 100 ft of arc) and 360° (angle subtended by a complete circle) is the same as the ratio between 100 ft of arc and the circumference of a circle having the same radius. Circumference equals $2\pi R$; therefore,

$$\frac{D}{360} = \frac{100}{2\pi R}$$

solving for R:

$$R = \frac{100 \times 360}{2\pi D}$$

$$R = \frac{5729.58}{D}$$

and also:

$$D = \frac{5729.58}{R}$$

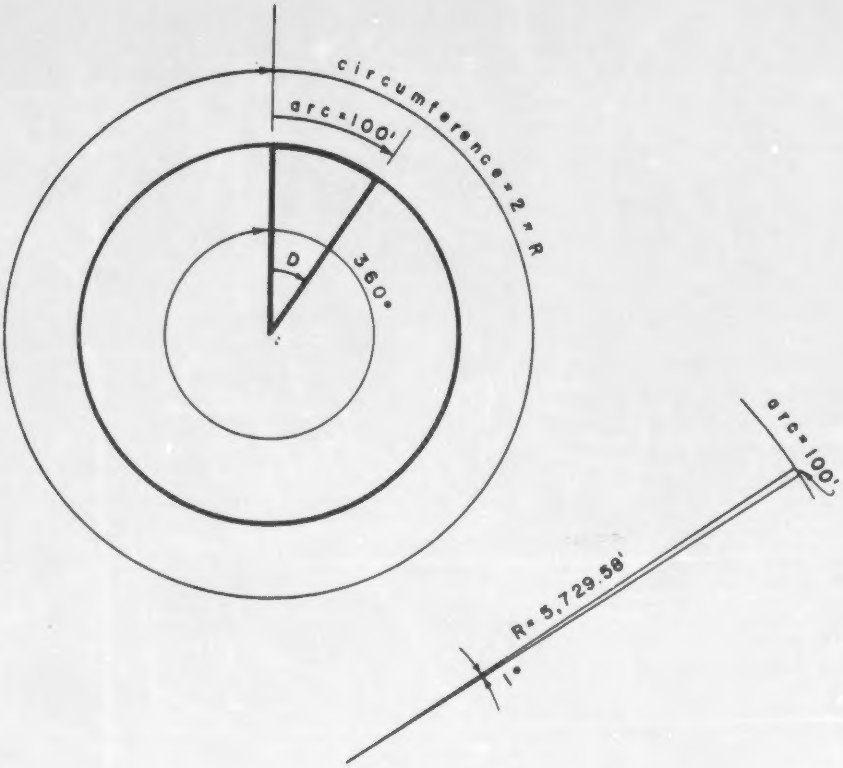

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Figure 8-6.—R and D, arc definition.

For a 1° curve, D = 1; therefore R = 5,729.58 ft.

In the case of the chord definition, figure 8-7, D subtends a 100-ft chord. In the right triangle,

$$\sin \frac{D}{2} = \frac{50}{R}$$

solving for R:

$$R = \frac{50}{\sin \frac{D}{2}}$$

For a 1° curve, D/2 is 0°30′, and the value for R computes to be 5,729.65 ft.

You will notice that the radii for 1° curves by the two definitions have very nearly the same value: 5,729.58 as opposed to 5,729.65. For some calculations not requiring great precision, the rounded-off value of 5,730 feet is used for the radius of a 1° curve.

Tangent Distance

If you study figure 8-8, you will see that the solution for T (tangent distance from P.C. or from P.T. to P.I.) is a simple right-triangle solution. T is one of the shorter sides of a right triangle; R is the other shorter side. T is the side opposite an angle which measures $\frac{\Delta}{2}$. Therefore,

$$\tan \frac{\Delta}{2} = \frac{T}{R}$$

and solving for T,

$$T = R \tan \frac{\Delta}{2}$$

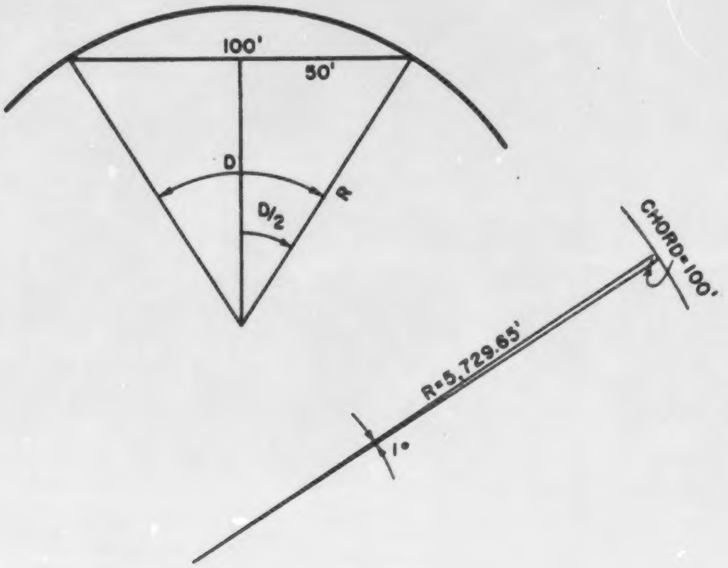

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Figure 8-7.—R and D, chord definition.

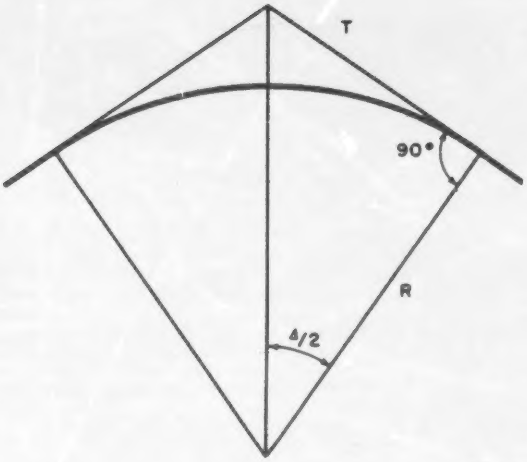

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Figure 8-8.—Tangent distance.

Chord Distance

The solution for the length of C is again a simple right-triangle solution. (See fig. 8-9.) One-half of C is one of the shorter sides of a right triangle; R is the hypotenuse of the same triangle. One-half C is the side opposite an angle equal to $\frac{\Delta}{2}$.

Therefore,

$$\sin \frac{\Delta}{2} = \frac{C/2}{R}$$

Solving for C:

$$C/2 = R \sin \Delta/2$$
$$C = 2R \sin \Delta/2$$

Middle Ordinate and External Distance

Two commonly used formulas for M and E are:

$$M = R\,(1 - \cos \Delta/2)$$
$$E = T \tan \Delta/4$$

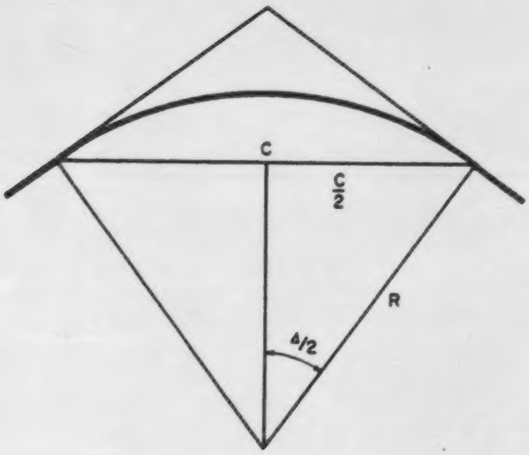

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Figure 8-9.—Chord distance.

Length of Curve

In the arc definition of the degree of curvature, length is measured along the arc. The relationship between L and a complete circle of the same radius, and Δ and the central angle of a complete circle (360°), may be expressed as (see fig. 8-10A)

$$\frac{L}{2\pi R} = \frac{\Delta}{360}$$

from which

$$L = \frac{2\pi R\Delta}{360}$$

Since there are 2π radians in a circle the number of radians corresponding to a given Δ are:

$$\frac{\Delta}{360} \times 2\pi$$

Note that in the equation $L = \frac{2\,\pi\,R\Delta}{360}$, the R is the only factor not included in the expression above for the number of radians. Therefore, the arc length may be expressed:

$$L = R \times \Delta \text{ in radians.}$$

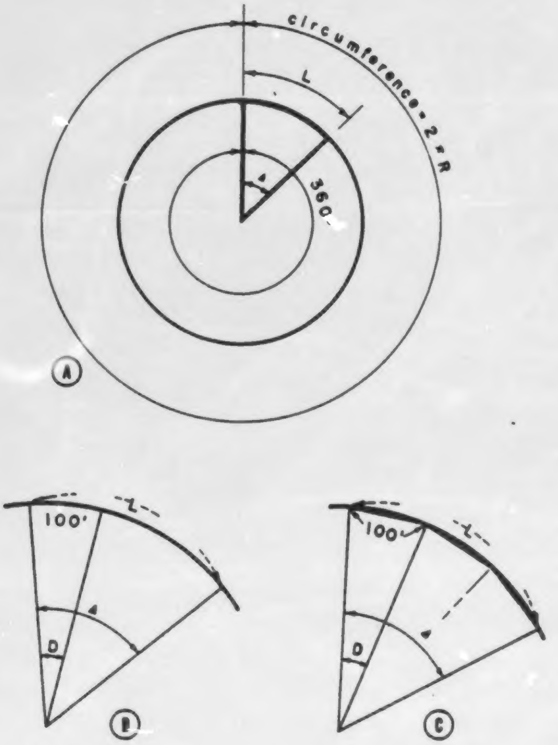

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Figure 8-10.—Length of curve.

This expression is frequently used to compute A by referring to a table of radians (sometimes called a "table of arc lengths for unit radius").

The relationship between D (see fig. 8-10B) and Δ and L and a 100-foot arc length may be expressed:

$$\frac{L}{100} = \frac{\Delta}{D}$$

from which

$$L = 100\frac{\Delta}{D}$$

This expression is also applicable to the chord definition (see fig. 8-10C). But L in this case is not the true arc length, because, in the expression $L = 100\,\frac{\Delta}{d}$ ' $\frac{\Delta}{D}$ indicates the number of 100-foot CHORD lengths and not the number of 100-foot ARC lengths.

Curve Areas

It is frequently necessary, when computing land areas, to determine certain areas bounded by elements of the curve. Two of these areas, which may be called the SEGMENTAL AREA (SA) and the EXTERNAL AREA (EA), are shown in figure 8-11.

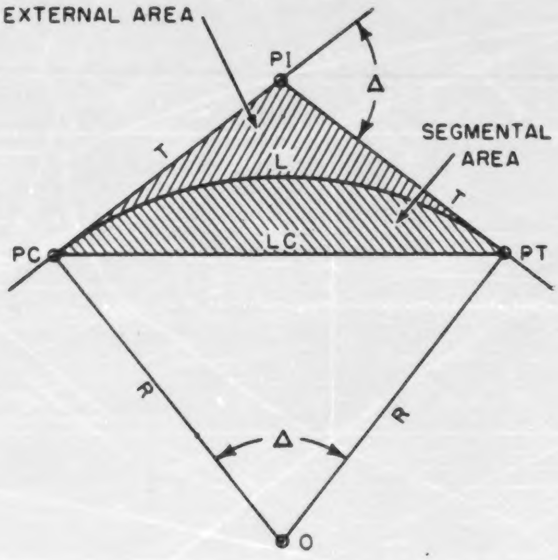

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Figure 8-11.—Curve areas.

Formulas which may be used to compute these areas are:

$$SA = R/2\,(A - R \sin \Delta)$$
$$EA = R\,(T - A/2)$$

A computational check may be made by comparing the sum of these two computations against the value determined by using the formula

$$SA + EA = C/2 \times C/2 \times \tan \Delta/2.$$

In order to get an exact check on your computations, you may have to use, for computational purposes, a greater number of significant digits for the elements (A, C, T, etc.) than you would normally need for stakeout.

An example of a computation of the areas follows.

GIVEN:
$R = 400.00$ ft.
$\Delta = 42°06'24''$
$C = 287.39$ ft.
$T = 153.973$ ft.
$L = 293.960$ ft.

TO FIND: EA, SA, and SA + EA

SOLUTION:
$SA = R/2\,(A - R \sin \Delta)$
$SA = 400.00/2\,(293.960 - (400.00 \times 0.670513)$
$SA = 200.00\,(293.960 - 268.205)$
$SA = 200.00\,(25.755)$
$SA = 5{,}151$ sq. ft.
$EA = R(T - L/2)$
$EA = 400.00\,(153.973 - (293.960/2)\,)$
$EA = 400.00\,(153.973 - 146.980)$
$EA = 400.00\,(6.993)$

$EA = 2{,}797$ sq. ft.

The sum of the two computations above is:

$$SA + EA = 5{,}151 + 2{,}797 = 7{,}948 \text{ sq. ft.}$$

To check the above:

$SA + EA = C/2 \times C/2 \times \tan \Delta/2$
$SA + EA = 287.39/2 \times 287.39/2 \times \tan 21°03'12''$
$SA + EA = 143.695 \times 143.695 \times 0.394932$
$SA + EA = 2064.825 \times 0.384932$
$SA + EA = 7.948$ sq. ft.

DEGREE OF CURVATURE COMPUTATIONS

The magnitude of the numerical differences between the arc and chord definitions of the degree of curvature is indicated in figures 8-12 and 8-13. In figure 8-12 you are given a curve of 12,277.70 ft. radius with a Δ of 56°30′. From tables (discussed later) you find that the D (chord def.) is 0°28′00″, which is 0.46666667° in terms of decimals of a degree. From before,

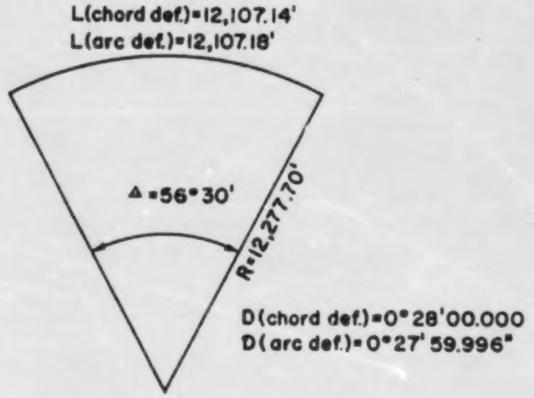

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Figure 8-12.—Example 1: Degree of curvature computations.

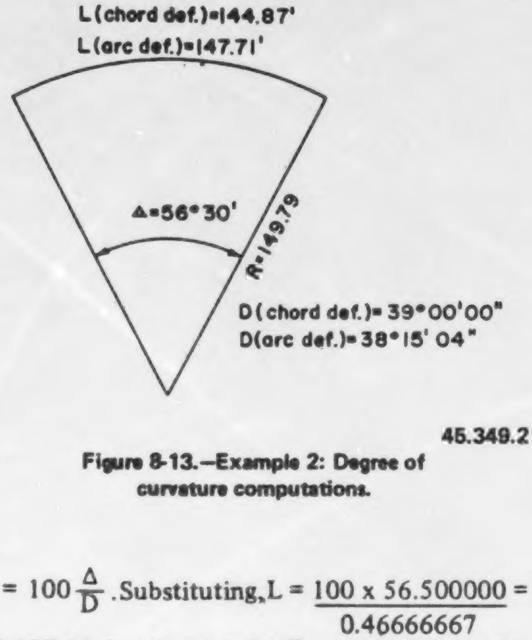

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Figure 8-13.—Example 2: Degree of curvature computations.

$L = 100\frac{\Delta}{D}$. Substituting, $L = \frac{100 \times 56.500000}{0.46666667} =$ 12,107.14 ft. (chord def.). For the length by the arc definition you may use $L = \frac{2\pi R\Delta}{360}$. Using the given radius, L computes to be 12,107.18 feet (arc def.). To find D (arc def.), you may use the relationship $L = 100\frac{\Delta}{D}$, which may be arranged $D = \frac{100\,\Delta}{L}$. Since you know Δ and L, you may substitute and find that D (arc def.) = 0.46666543° or 0°27'59.996". As you can see, there is very little difference between the L's and D's for a curve of such long radius.

Now refer to figure 8-13. Again you are given a Δ of 56°30'. But you are now given a short radius of 149.79 feet. From tables based on the chord definition, you may determine that D (chord def) = 39°00'00". Using $L = 100\frac{\Delta}{D}$, you find L (chord def.) = 144.87 feet. Using the same procedures as for the long radius curve, you find D (arc. def.) = 38°15'04" and L (arc def.) = 147.71 feet. Compare the D's and L's and you will see that there is a marked difference.

In these examples, the Δ and R were held in each case, and the D and L computed for each definition. The same magnitude of differences may be shown by holding any two of the elements (except, of course, R and D) and computing the others. From this demonstration you should learn two important facts. First, that you must always determine whether arc or chord definition is intended when D is specified. Second, that the numerical differences between the arc and chord definitions are much more significant on short radius curves than on long radius curves.

PLOTTING AND STATIONING A CURVE

You now have all the formulas you need to make a basic analysis and a plot of a horizontal curve. The data you have given is the location of the P.I., the size of Δ (the central angle), and the degree of curvature (D). We'll assume that the degree of curvature (arc definition) is 2°00', that the location of the P.I. is station 75 + 15.12, and that the size of Δ is 45°30'.

Begin by plotting the tangents from the P.I. as illustrated in figure 8-14. Draw one of the tangents at a convenient angle on the paper, and draw the other at a deflection angle of 45°30'. Mark the point of intersection (P.I.) with the station (75 + 15.12).

The next step is to locate the P.C. and the P.T. by computing T. To compute T, you must first compute R, which is 5,729.58/D, or 5,729.58/2, or 2,864.79 ft. T equals 2,864.79

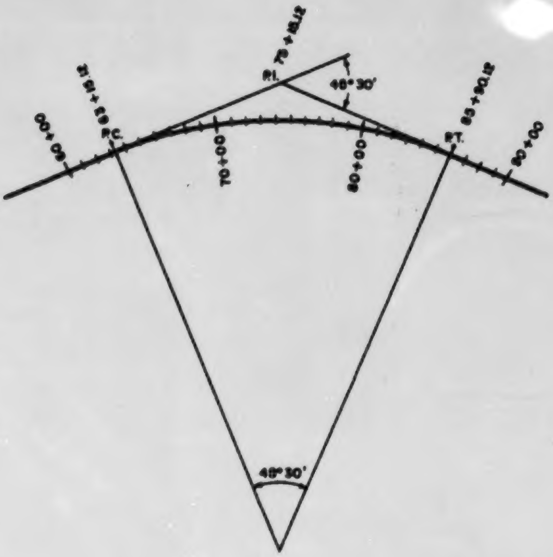

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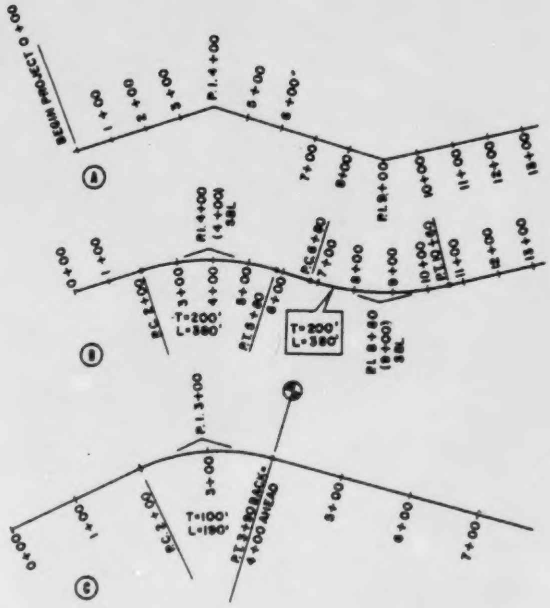
Figure 8-14.–Plotting a curve.

tan 1/2 Δ, or 2,864.79 tan 22.75°, or (by slide rule) 1200 ft. For more precise values, you will have to use logs or a calculating machine.

Select a suitable scale on the engineer's scale, and measure off 1200 ft on the tangents from the P.I. This locates the P.C. and the P.T. Draw the radii from the P.C. and the P.T. by erecting perpendiculars to the tangents. Each radius should scale 2,864.79 ft. and the angle between them should measure 45°30′. If this is not the case, you have made a mistake somewhere.

Compute and set down the stations of the P.C. and P.T. The station of the P.C. is (7515.12 - 1200), or 6315.12, or 63 + 15.12. The station of the P.T., however, is DEFINITELY NOT (7515.12 + 1200), or 8715.12, or 87 + 15.12, and you must avoid the common mistake of assuming that it is. The distance from the P.C. to the P.T. is the LENGTH OF THE CURVE or L. To compute the station of the P.T., you must compute L and add it to the station of the P.C. L equals 100 Δ/D or 100 x 45.5/2 or 2275.00 ft. The station of the P.T., then, is (6315.12 + 2275.00), or 8590.12, or 85 + 90.12.

In highway work, the SURVEY BASE LINE, which is run in the field before the curves are designed, is generally stationed as shown in figure 8-15, example A. After the curves are computed, new stationing is set on the centerline, using the lengths of the curves in the manner just described. The new P.I. stations are based on stationing back, not ahead. Reference may be made on the plans to the original SBL stations of the P.I.'s as shown in figure 8-15, example B. In some cases, where there are few curves, for example, you may find it desirable to use the original stationing on the tangents and establish an EQUALITY at the P.T., as shown in figure 8-15, example C. At an equality, the stationing value for the line back is different from the stationing value of the line ahead. In the illustration, the difference between the L and the sum of the T's is reflected in the difference of the stationing back and ahead. Equalities are also used to correct for revisions to the centerline alignment after the final stationing has been set, and also to correct for errors or mistakes in fieldwork or computations. An equality is sometimes marked on the plan with a distinctive symbol to help direct attention to the change in stationing.



45.351

Figure 8-15.–Stationing curves.

TABLES OF CURVE FUNCTIONS

Many of the elements of curves you have computed may be obtained more easily, particularly in the field, from tables.

Radii

Figure 8-16 shows a page from a table of RADII AND THEIR LOGARITHMS. This table contains the radii and their logarithms for curves of every degree of curvature, chord definition, to the nearest minute, from 0° to 40°. As you can see, the radius for the 2-degree curve used in the last section is listed as 2864.93 ft; for the arc definition, we got 2864.79 ft. Next to the radius is the log of the radius.

Functions of a 1° Curve-Arc Definition

Figure 8-17 shows a page from a table of FUNCTIONS OF A 1° CURVE, arc definition, using R = 5730. This table lists, under central angles to the nearest minute from 0° to 106°, the L.C., M, E, and the T of a 1° curve. To get the L.C., M, E, or T for a curve other than a 1° curve, you divide the listed value by D, the degree of curvature. For us, D equals 2°. Under 45° and beside 30′ the listed T is 2402.8. This is T for a 1° curve with a Δ of 45°30′. For our 2° curve, T equals 2402.8/2, or 1201.4 ft.

The explanation of why dividing a tabular value by your given degree of curvature (D) will give you the value for that particular curve is as follows. You know that, for any given horizontal curve, $T = R \tan 1/2\ \Delta$, and $R = 5730/D$. Therefore, substituting 5730 for R in the first equation, $T = \frac{5730 \times \tan 1/2\Delta}{D}$. The values given for T in the table are derived from this formula, with a D in each case of 1°. For example: for a curve with a central angle (Δ) of 5°, the listed value of T is 250.17. If you work out the formula $\frac{5730 \tan 2.5^\circ}{1}$ you will find that the result is 250.17. You can see that, to get the value for a D of other than 1°, you would simply substitute that D for the 1 in the formula. To divide the result of the formula by the specific D amounts to the same thing.

The explanation with regard to the L.C. is similar. For any horizontal curve, $L.C. = 2R \sin 1/2\ \Delta$. Substituting $2(5730/D)$ for 2R, $L.C. = \frac{11460 \sin 1/2\Delta}{D}$ The values in the table are derived from this formula, with a D in each case of 1°. To convert a tabular value to a value for a D of other than 1°, you simply divide the tabular value by the specific D.

The procedure for the determination of M and E are, again, similar. For a curve of given Δ, divide the tabular M or E by the given D to obtain the final value of M or E.

Precision of Computations

The values in the table of functions of a 1° curve shown in figure 8-17 are based on a 1° curve having a radius of the rounded-off value of 5730 feet instead of the more precise value of 5729.58 feet (arc definition) presented earlier in the chapter. This difference won't matter for many types of computations, but there are times when you may be using numbers with so many significant figures that the table will not provide the proper precision. For example, the curve in figure 8-12 has a radius of 12,277.7 feet (to six significant figures) and a Δ of 56°30′ Using the formula $L.C. = 2R \sin \Delta/2$ you get L.C. = 11,622.6 feet. But, using the table, you enter under $\Delta = 56^\circ 30'$ (not shown in the figure) and get L.C. = 5424.1 for the 1° curve (based on an R rounded off at the fourth significant figure). Divide by D (which is 0.466665°) and you get L.C. = 11,623.1 feet; a difference from using the formula of 0.5 foot–too much for some types of computations. This example points out the necessity of being aware of the number of significant figures involved in your calculations. For most fieldwork, however, tables such as those shown will give you answers of sufficient precision.

Functions of a 1° Curve–Chord Definition

In the table shown in figure 8-17, the values of L.C., M, E, and T are exact for the arc definition only, not the chord definition. This is

Deg. D.	Radius R.	Log. R.	Deg. D.	Radius R.	Log. R.	Deg. D.	Radius R.	Log. R.
° ′			° ′			°		
0 0	∞	∞	1 0	5729.65	3.758128	2 0	2864.93	3.457115
1	343774.68	5.536274	1	5635.72	.750950	1	2841.26	.453511
2	171887.34	.235244	2	5544.83	.743888	2	2817.97	.449937
3	114591.56	.059153	3	5456.82	.736939	3	2795.06	.446392
4	85943.67	4.934214	4	5371.56	.730100	4	2772.53	.442876
5	68754.94	4.837304	5	5288.92	3.723367	5	2750.35	3.439388
6	57295.79	.758123	6	5208.79	.716737	6	2728.52	.435928
7	49110.68	.691176	7	5131.06	.710206	7	2707.04	.432496
8	42971.84	.633184	8	5055.59	.703772	8	2685.89	.429089
9	38197.20	.582031	9	4982.33	.697432	9	2665.08	.425710
10	34377.48	4.536274	10	4911.15	3.691183	10	2644.58	3.422356
11	31252.26	.494881	11	4841.96	.685023	11	2624.39	.419029
12	28647.90	.457093	12	4774.74	.678949	12	2604.51	.415727
13	26444.22	.422331	13	4709.33	.672959	13	2584.93	.412449
14	24555.35	.390146	14	4645.69	.667051	14	2565.65	.409197
15	22918.33	4.360183	15	4583.75	3.661221	15	2546.64	3.405968
16	21485.94	.332154	16	4523.44	.655469	16	2527.92	.402763
17	20222.06	.305825	17	4464.70	.649792	17	2509.47	.399582
18	19098.61	.281002	18	4407.46	.644189	18	2491.29	.396424
19	18093.43	.257521	19	4351.67	.638656	19	2473.37	.393289
20	17188.76	4.235244	20	4297.28	3.633194	20	2455.70	3.390176
21	16370.25	.214055	21	4244.23	.627799	21	2438.29	.387085
22	15626.15	.193852	22	4192.47	.622470	22	2421.12	.384016
23	14946.75	.174547	23	4141.96	.617206	23	2404.19	.380969
24	14323.97	.156064	24	4092.66	.612005	24	2387.50	.377943
25	13751.02	4.138335	25	4044.51	3.606866	25	2371.04	3.374938
26	13222.13	.121302	26	3997.48	.601787	26	2354.80	.371954
27	12732.43	.104911	27	3951.54	.596766	27	2338.78	.368990
28	12277.70	.089117	28	3906.64	.591803	28	2322.98	.366046
29	11854.33	.073877	29	3862.74	.586896	29	2307.39	.363122
30	11459.19	4.059154	30	3819.83	3.582044	30	2292.01	3.360217
31	11089.54	.044914	31	3777.85	.577245	31	2276.84	.357332
32	10743.00	.031125	32	3736.79	.572499	32	2261.86	.354466
33	10417.45	.017762	33	3696.61	.567804	33	2247.08	.351618
34	10111.06	.004797	34	3657.29	.563160	34	2232.49	.348789
35	9822.18	3.992208	35	3618.80	3.558564	35	2218.09	3.345979
36	9549.34	.979973	36	3581.10	.554017	36	2203.87	.343187
37	9291.25	.968074	37	3544.19	.549517	37	2189.84	.340412
38	9046.75	.956492	38	3508.02	.545063	38	2175.98	.337655
39	8814.78	.945212	39	3472.59	.540654	39	2162.30	.334915
40	8594.42	3.934216	40	3437.87	3.536289	40	2148.79	3.332193
41	8384.80	.923493	41	3403.83	.531968	41	2135.44	.329488
42	8185.16	.913027	42	3370.46	.527690	42	2122.26	.326799
43	7994.81	.902808	43	3337.74	.523453	43	2109.24	.324127
44	7813.11	.892824	44	3305.65	.519257	44	2096.39	.321471

45.45.4

Figure 8-16.—Sample page from table of radii and their logarithms.

	44°				45°				
	L. C.	M.	E.	T.	L. C.	M.	E.	T.	
0	4293.0	417.2	450.0	2315.1	4385.5	436.2	472.1	2373.4	0
2	4296.1	417.8	450.7	2317.0	4388.6	436.8	472.9	2375.4	2
4	4299.2	418.4	451.5	2319.0	4391.7	437.5	473.6	2377.3	4
6	4302.2	419.1	452.2	2320.9	4394.7	438.1	474.4	2379.3	6
8	4305.3	419.7	452.9	2322.8	4397.8	438.8	475.1	2381.2	8
10	4308.4	420.3	453.7	2324.8	4400.9	439.4	475.9	2383.2	10
12	4311.5	421.0	454.4	2326.7	4404.0	440.0	476.6	2385.2	12
14	4314.6	421.6	455.1	2328.7	4407.0	440.7	477.4	2387.1	14
16	4317.7	422.2	455.9	2330.6	4410.1	441.3	478.1	2389.1	16
18	4320.7	422.9	456.6	2332.6	4413.2	442.0	478.9	2391.0	18
20	4323.8	423.5	457.3	2334.5	4416.3	442.6	479.6	2393.0	20
22	4326.9	424.1	458.1	2336.4	4419.3	443.2	480.4	2394.9	22
24	4330.0	424.8	458.8	2338.4	4422.4	443.9	481.1	2396.9	24
26	4333.1	425.4	459.5	2340.3	4425.5	444.5	481.9	2398.8	26
28	4336.2	426.0	460.3	2342.3	4428.6	445.2	482.6	2400.8	28
30	4339.2	426.7	461.0	2344.2	4431.6	445.8	483.4	2402.8	30
32	4342.3	427.3	461.7	2346.1	4434.7	446.4	484.2	2404.7	32
34	4345.4	427.9	462.5	2348.1	4437.8	447.1	484.9	2406.7	34
36	4348.5	428.6	463.2	2350.0	4440.9	447.7	485.7	2408.6	36
38	4351.6	429.2	463.9	2352.0	4444.0	448.3	486.5	2410.6	38
40	4354.7	429.8	464.7	2353.9	4447.0	448.9	487.2	2412.6	40
42	4357.7	430.5	465.4	2355.9	4450.1	449.5	488.0	2414.5	42
44	4360.8	431.1	466.2	2357.8	4453.2	450.2	488.7	2416.5	44
46	4363.9	431.7	466.9	2359.8	4456.3	450.8	489.5	2418.5	46
48	4367.0	432.4	467.7	2361.7	4459.3	451.5	490.3	2420.4	48
50	4370.1	433.0	468.4	2363.7	4462.4	452.1	491.0	2422.4	50
52	4373.2	433.6	469.1	2365.6	4465.5	452.7	491.8	2424.4	52
54	4376.2	434.3	469.9	2367.6	4468.6	453.4	492.5	2426.3	54
56	4379.3	434.9	470.6	2369.5	4471.6	454.1	493.3	2428.3	56
58	4382.4	435.6	471.4	2371.5	4474.7	454.8	494.1	2430.2	58
60	4385.5	436.2	472.1	2373.4	4477.8	455.5	494.8	2432.2	60

45.45.5

Figure 8-17.—Sample page from table of functions of a 1° curve.

because in the case of the arc definition, R varies inversely as D, and you reflect this relationship when you divide the tabular values (which are functions of R) by D. But for the chord definition, R varies inversely with the sin D/2 instead of with D. The net result is, that for the chord definition, a small correction must be applied to the tabular values obtained. Figure 8-18 shows a table which provides such corrections for T and E. Note that the corrections become larger as both Δ and D increase.

The differences involved may be demonstrated by assuming D = 30° for both an arc definition curve and a chord definition curve. Using the formulas $R = \frac{5729.58}{D}$ and $R = \frac{50}{\sin D/2}$ or tables based on these formulas, you may determine that R for the arc definition is 190.99 feet and R for the chord definition is 193.19 feet; all as shown in figure 8-19. Now assume that Δ is 45° in both cases. Using the formula $T = R \tan \Delta/2$, you get T (arc def.) = 79.11, and T (chord def.) = 80.02. Now compare these values with those determined by using the table shown in figure 8-17. Under T for 45°00′ you will find the value 2373.4 ft. for the 1° curve. Divide the 2373.4 by 30 (which is D) and you get 79.11 feet. Note that this agrees with the value for T

Corrections To Be Added to Tangents and External Distances for Curves Laid Out by Chord Definition

FOR TANGENTS ADD

Central angle (degrees)	Degree of curve								
	5°	10°	15°	20°	25°	30°	40°	50°	60°
10	0. 03	0. 06	0. 09	0. 13	0. 16	0. 19	0. 25	0. 31	0. 38
20	. 06	. 13	. 19	. 26	. 32	. 39	. 51	. 65	. 79
30	. 10	. 19	. 29	. 39	. 49	. 59	. 79	. 99	1. 20
40	. 13	. 26	. 40	. 53	. 67	. 80	1. 06	1. 34	1. 64
50	. 17	. 34	. 51	. 68	. 85	1. 02	1. 36	1. 72	2. 10
60	. 21	. 42	. 63	. 84	1. 05	1. 27	1. 71	2. 17	2. 60
70	. 25	. 51	. 76	1. 02	1. 28	1. 54	2. 06	2. 60	3. 16
80	. 30	. 61	. 91	1. 22	1. 53	1. 84	2. 46	3. 10	3. 78
90	. 36	. 72	1. 09	1. 45	1. 83	2. 20	2. 94	3. 70	4. 50
100	. 43	. 86	1. 30	1. 74	2. 18	2. 62	3. 50	4. 40	5. 37
120	. 62	1. 25	1. 93	2. 52	3. 16	3. 81	5. 11	6. 44	7. 80

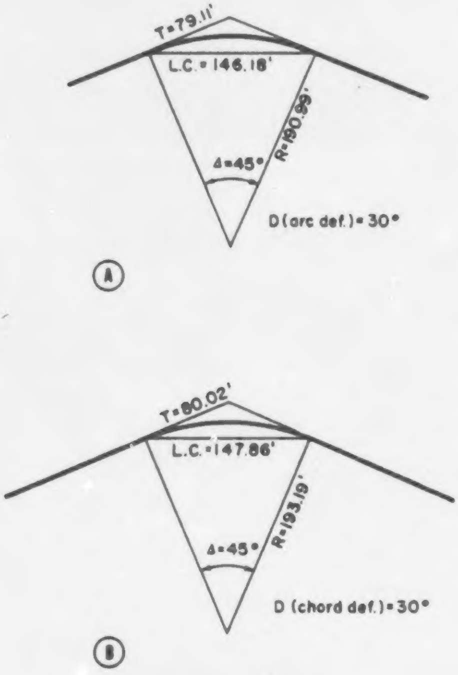
FOR EXTERNALS ADD

Central angle (degrees)	Degree of curve								
	5°	10°	15°	20°	25°	30°	40°	50°	60°
10	0. 001	0. 003	0. 004	0. 006	0. 007	0. 008	0. 011	0. 014	0. 017
20	. 006	. 011	. 017	. 022	. 028	. 034	. 045	. 057	. 070
30	. 013	. 025	. 038	. 051	. 065	. 078	. 103	. 129	. 170
40	. 023	. 046	. 070	. 093	. 117	. 141	. 203	. 265	. 290
50	. 037	. 075	. 116	. 151	. 189	. 227	. 305	. 384	. 467
60	. 056	. 112	. 168	. 225	. 283	. 340	. 457	. 575	. 697
70	. 080	. 159	. 240	321	. 403	. 485	. 652	. 819	. 994
80	. 110	. 220	. 332	. 445	. 558	. 671	. 903	1. 13	1. 38
90	. 149	. 299	. 450	. 603	. 756	. 910	1. 22	1. 54	1. 87
100	. 200	. 401	. 604	. 809	1. 01	1. 22	1. 64	2. 06	2. 50
120	. 360	. 721	1. 08	1. 45	1. 82	2. 19	2. 95	3. 72	4. 50

45.45.6

Figure 8-18.—Table of corrections to be added to tangents and external distances for curves laid out by chord definition.

(arc definition) worked out from the formula. This value must be corrected if you are using the chord definition, so enter the table in figure 8-18 under a 30° curve and opposite the 40° and 50° angles. Interpolate (for the 45° Δ assumed in this case) between the values 0.80 and 1.02 shown, and you get 0.91 ft. correction. Add this correction to the 79.11 feet you get from the table in figure 8-17, and your answer is 80.02. This agrees with the values determined from the formula. To work out the values for L.C. you may use the formula L.C. = 2R sin Δ/2, using the R computed for each definition. The results are shown in figure 8-19. Now compare these with the L.C. computed by using the table in figure 8-17. Under 45°00′ you will find L.C. for the 1° curve is 4385.5 feet; divide by 30 (which is D) and you get 146.18 feet. This agrees with the value shown in 8-19 for the arc definition. Figure 8-18 shows corrections for E as well as T; the procedure for computing E is the same as that described for computing T.



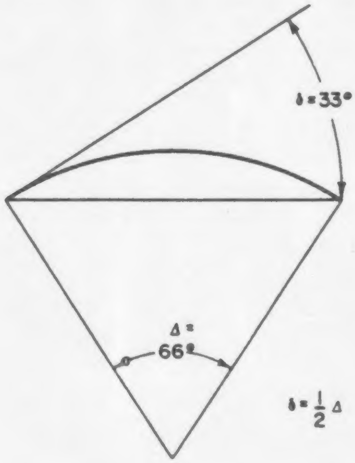
45.352

Figure 8-19.—Curve computations: arc definition versus chord definition.

PLOTTING BY DEFLECTION ANGLES

You could plot in the curve in figure 8-14 by compass, providing you had a compass large enough to be spread to the radius of the curve. However, we'll plot the curve in by the DEFLECTION ANGLE method, because this is the method most frequently used to stake out a curve in the field.

First, consider figure 8-20. This figure shows an arc, a tangent drawn at one end of the arc, and the chord of the arc. The arc subtends an angle (the one we call the central angle, or Δ) of 66°. The angle between the tangent and the chord (called δ, or little delta) measures one-half of that, or 33°. The deflection-angle method of curve layout is based on the fact that the angle formed by a tangent drawn at one end of an arc and the chord of the arc (that is, the deflection angle of the chord from the tangent) is one-half the size of the angle subtended by the arc.



45.353

Figure 8-20.—Angle between tangent and chord equals one-half of central angle.

We'll apply that fact, now, to plot a series of points along the arc of the curve illustrated in figure 8-21. You are given D = 20° (arc def.) and Δ = 80°. First draw a line for the back tangent, select a point on the line for the P.I., protract the Δ of 80°, and draw the tangent ahead. Next, determine T using a table or formula. To use the formula you need R:

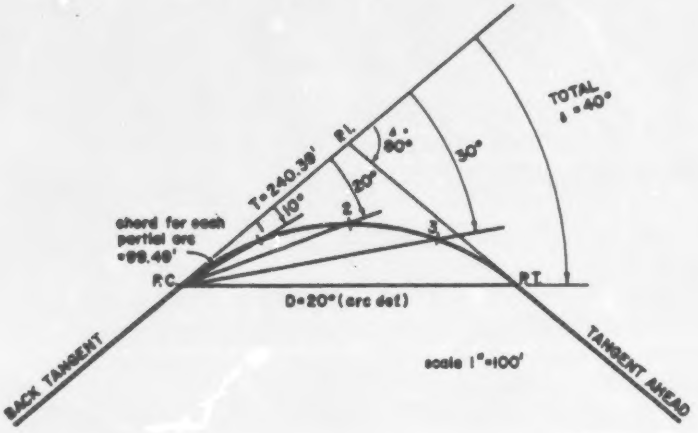
$$
\begin{aligned}
D &= 20^\circ \text{ (arc def.)}\\
R &= 5729.58/D\\
R &= 5729.58/20\\
R &= 286.48 \text{ ft.}
\end{aligned}
$$

Using R, you may compute T:

$$
\begin{aligned}
T &= R \tan \Delta/2\\
T &= 286.48 \times \tan\ (80^\circ/2)\\
T &= 286.48 \times \tan 40^\circ\\
T &= 286.48 \times 0.83910\\
T &= 240.39 \text{ ft.}
\end{aligned}
$$

Scale T back from the P.I. and mark the P.C.; scale T ahead of the P.I. and mark the P.T.

Now assume that you decide to plot three points on the curve between the P.C. and the P.T., dividing the arc into four equal segments. The Δ for the total curve is 80°; each segment is 1/4 of the total arc; therefore the Δ for each



45.354

Figure 8-21.–Plotting a curve by drawing a succession of chords.

partial arc is 80°/4 or 20°. Since the deflection angle is 1/2 80° or 40°, and δ for each segment is 1/2 20° or 10°. Set the center of your protractor at the P.C. with 0° on the line through the P.I.; mark off deflections of 10°, 20°; 30°; and 40°; then draw four rays from the P.C. The three intermediate points on the curve will be on the first three rays; and the last ray, passing through the P.T., marks the long chord of the whole curve. The final step in locating the P.O.C.'s is to set your compass point at the P.C. and mark an arc (numbered 1 in figure 8-21) across the first ray; the intersection of the arc and the ray is the first P.O.C. Then shift your compass point to the first P.O.C. and swing arc 2, marking the second P.O.C. Swing arc 3 from the second P.O.C. in the same manner. The last arc, swung from the third P.O.C., should intersect the P.T. BUT, what length do you set your compass to when you swing the arcs? You use the chord which subtends each partial arc. In this case you divided the curve into four equal parts, each with a Δ of 20°. The chord distance to which you set your compass, then, is:

$$\text{chord} = 2R \sin \Delta/2$$
$$\text{chord} = 2(286.48) \times \sin (20°/2)$$
$$\text{chord} = 572.96 \times \sin 10°$$
$$\text{chord} = 572.96 \times 0.17365$$
$$\text{chord} = 99.49 \text{ ft.}$$

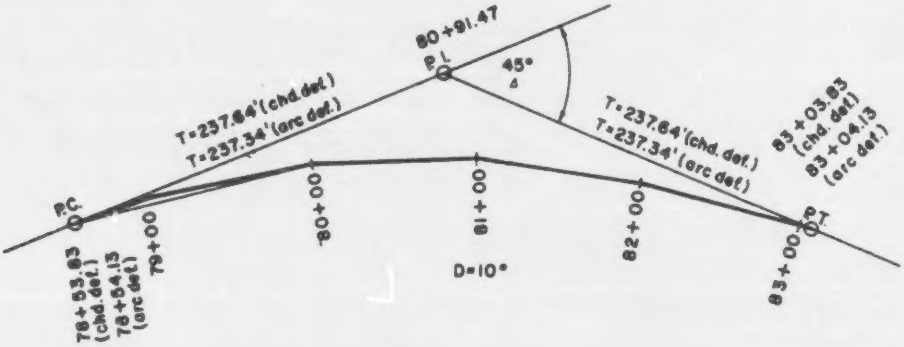
After you have located the P.O.C.'s you can draw in the line of the arc with a railroad curve or a spline.

CURVE STAKEOUT IN THE FIELD

The foregoing is, basically, the method used to stake out a curve in the field. The transit is used to turn the deflection angles, and the steel tape is generally used to lay off chords. Field stakeout, however, is complicated by the fact that the curve must be correctly stationed along its length, and the lengths of the chords laid out in the field are selected to meet the requirements of the job. Also, it may not be possible to turn all the deflection angles from the setup at the P.C., so you may have to move ahead on the curve.

Staking the P.C. and P.T.–Arc Definition

Suppose that you are setting the alignment and staking the centerline of a road in the field, that you have set a P.I. at station 80 + 91.47 and established a Δ of 45°, and that you have decided upon a 10° curve (arc definition); all as shown in figure 8-22. Reference to a table of functions of a 1° curve shows that, for a Δ of 45°, T is 2373.4 for a 1° curve Divide by 10 (since D is 10°) and you find that T is 237.34 ft. Subtract T from the station of the P.I. and you get the station of the P.C.: 78 + 54.13.



45.355

Figure 8-22.—Staking a curve.

The next step is to determine L, so that you may station the P.T. Since $L = 100\frac{\Delta}{D}$, L in this case is $100 \times \frac{45}{10}$ or 450.00 feet. Add 450.00 feet to the station of the P.C. (78 + 54.13) and you get the station of the P.T.: 83 + 04.13.

Measure 237.34 feet back from the P.I. and mark the P.C.; measure 237.34 feet ahead from the P.I. and mark the P.T. Staking the P.T. gives you a point to check in to when you perform the final operation, that is, staking the points on the curve.

Staking the P.C. and P.T.—Chord Definition

If you had selected a 10° curve, chord definition (instead of arc definition) in the preceding example, you would follow exactly the same procedure in computing and staking the P.C. and P.T. But the T is somewhat different for the chord definition since the correction must be applied. The table in figure 8-18 indicates, under a 10° curve and interpolating for the 45° central angle between the values opposite 40° and 50°, a correction of 0.30 ft. Therefore, the T for the chord definition is 237.34 ft plus 0.30 ft or 237.64 ft. Subtracting T from the station of the P.I. (80 + 91.47) you get station 78 + 53.83 for the P.C. (See fig. 8-22.) The length is still 450.00 ft, but for the chord definition it represents the distance along a series of 100-foot chords instead of the distance along the arc. The procedure for stationing the P.T. is the same as before: add the L (450.00 ft) to the station of the P.C. (78 + 53.83) and you get 83 + 03.83, the station of the P.T.

Staking the P.O.C.'s

After the P.C. and P.T. have been staked, the next step is to stake the selected stations on the curve. The usual procedure is to occupy the P.C. and turn the deflection angles to successive stations while the chords are laid off from one station to the next. Set up at the P.C., set the plate at zero, back sight at a point on the back tangent, plunge the scope, and turn the deflections. Or, if it will give you a longer backsight, use the P.I. as the backsight. To set the first P.O.C., one chainman holds the end of the tape at the instrument (at the P.C.) while the second chainman holds the appropriate chord distance on the tape and swings the tape until he intersects the line of sight of the first deflection angle; he then marks a stake with the station number and drives a hub and/or stake at the point established. Next, the instrumentman turns the second deflection, the chainmen move ahead to the second P.O.C., and the second station is marked; the process is repeated at each point until the P.T. is reached—the last angle and distance should check in to the previously-staked P.T.

The stations may be staked at intervals of 100, 50, 25, or sometimes 10 feet, depending on the type of construction and the degree of curvature. The intervals are marked at full stations and fractions thereof, su[illegible]0, 9 + 25, 9 + 50, 9 + 75, 10 + 00, and s[illegible]n. If a curve were to begin and end at full stations, and you were staking it on 100-foot stations, the computations would be simple; the chords would all be the same, and deflections would all be the same (each angle would be D/2, because δ is 1/2 Δ, and Δ is D for a 100-foot length). But P.C.'s and P.T.'s almost always fall at odd pluses, that is, beyond a station at a plus distance other than the specified interval. Therefore, you will usually have to figure an irregular deflection and chord for the first length past the P.C. and for the last length before the P.T.

Chords–Arc Definition

When the arc definition of the degree of curvature is specified, the length of a curve is stationed along the arc; but, in the field, you must measure chords, and not arcs, with the tape. The chord is always shorter than the arc; and the problem created by this fact may be solved in either of two ways. First, you may use chords that are short enough so that the difference between the arc and chord lengths is insignificant; or second, you may determine and measure the actual chord length which corresponds to the arc length being stationed.

To eliminate corrections by using short chords, you must consider the degree of curvature. The sharper the curve, the shorter the chords must be to obtain the precision appropriate to the given job. For work of ordinary precision, a typical guide which might be established for a job is:

For D of less than 2°, use 100-ft chords
For D from 2° to 7°, use 50-ft chords
For D from 7° to 18° use 25-ft chords
For D greater than 18°, use 10-ft chords.

To determine the chord length for a given arc length, you may either compute the chord using the formulas presented earlier in this chapter, or refer to tables which show chord lengths for arc definition curves. Figures 8-23 and 8-24 show such tables; one is for curves defined by D, and the other for curves defined by R.

Now apply the foregoing information to the 10° curve shown in figure 8-22. Remember that

Degree of curve (degrees)	Radius=5,729.58 $\frac{(1)}{D}$	Chord for 25 feet of arc	Chord for 50 feet of arc	Chord for 100 feet of arc
1	5, 729. 58	25. 00	50. 00	100. 00
2	2, 864. 79	25. 00	50. 00	100. 00
3	1, 909. 86	25. 00	50. 00	99. 99
4	1, 432. 40	25. 00	50. 00	99. 98
5	1, 145. 92	25. 00	50. 00	99. 97
6	954. 93	25. 00	50. 00	99. 95
7	818. 51	25. 00	50. 00	99. 94
8	716. 20	25. 00	49. 99	99. 92
9	636. 62	25. 00	49. 99	99. 90
10	572. 96	25. 00	49. 98	99. 87
12	477. 46	25. 00	49. 98	99. 82
14	409. 26	25. 00	49. 97	99. 75
16	358. 10	25. 00	49. 96	99. 68
18	318. 31	24. 99	49. 95	99. 59
20	286. 48	24. 99	49. 94	99. 49
22	260. 44	24. 99	49. 92	99. 39
24	238. 73	24. 99	49. 91	99. 27

45.45.7

Figure 8-23.–Table of chord lengths (for degree of curve by arc definition).

Radius	Chords for 25' arcs	Radius	Chords for 100' arcs
100	24.93	600	99.88
150	24.97	700	99.91
200	24.98	800	99.93
		900	99.95
	Chords for 50' arcs	1,000	99.96
		1,500	99.97
		2,000	99.98
250	49.92	2,500	99.99
300	49.94	3,000	100.00
350	49.96		
400	49.97		
450	49.97		
500	49.98		

45.45.8

Figure 8-24.—Table of lengths of chords for even radius curves (arc definition).

Curve Stakeout Table
(Arc Definition)

D=10° L=450.00 Δ=45°

Station	Arc	Δ	δ	Total Deflection	Chord
P.C.-78+54.13					
	45.87	4°35'	2°17'30"		45.85
79+00				2°17'30"	
	100.00	10°00'	5°00'00"		99.87
80+00				7°17'30"	
	100.00	10°00'	5°00'00"		99.87
81+00				12°17'30"	
	100.00	10°00'	5°00'00"		99.87
82+00				17°17'30"	
	100.00	10°00'	5°00'00"		99.87
83+00				22°17'30"	
	4.13	0°25'	0°12'30"		4.13
P.T.-83+04.13				22°30'00"	
	450.00	45°00'	22°30'		

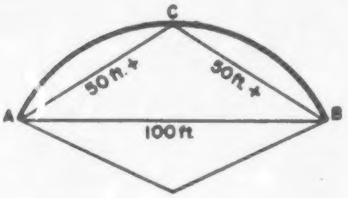
45.356

Figure 8-25.—Curve stakeout table (arc definition).

you are using the arc definition for this first example. You may find it convenient to tabulate the stakeout information as shown in figure 8-25. The first column shows the stations. The second shows the arc lengths between the stations; these are found by subtracting one station from the next. The totals at the bottom of three of the columns are a check on your arithmetic. The third column shows the central angle for each arc length. By definition, the Δ for a 100-foot length is 10° because D is 10°. The other central angles are proportional: the Δ for 45.87 ft is $\frac{45.87}{100} \times 10^\circ$ or 4.587° or 4°35′; the Δ for 4.13 ft is $\frac{4.13}{100} \times 10^\circ$ or 0.413° or 0°25′. The deflection angles in the fourth column are as previously explained, one-half the central angles. The fifth column shows the total of the deflection angles to the station indicated; these are the values which you will set on your instrument as the chainmen progress from one station to the next along the curve. The last column shows the chords which the chainman will measure from one station to set the next. Reference to the table in figure 8-23 shows that the chord for a 50-foot arc of a 10° curve is 49.98; that is, a 0.02 ft correction. The first arc length in the table (45.87 ft) is sufficiently close to 50 ft, so the same correction is reasonable; therefore, the first chord is 45.87 minus 0.02 or 45.85 ft. The reasonableness of this procedure may be shown by using the formula $C = 2R \sin \Delta/2$ and a more precise value of Δ $C = 2(572.96)(\sin 2^\circ 17'36'') = (1145.92)(0.040016) = 45.855$ ft; sufficiently close to 45.85 ft for most work. The chords for the 100-foot arcs are 99.87 ft, from the table. The last arc, 4.13 ft, is so short that no correction is needed.

Chords—Chord Definition

When the chord definition of the degree of curvature is specified, the length of the curve is stationed along a series of 100-foot chords, which is what you may be actually measuring in the field. But when the P.C. and P.T. fall at odd pluses, and when the curve is sharp, you will be laying off chords of less than 100 ft; and these subchords will be LONGER than their nominal proportion of the standard 100-foot chord. This fact may be readily seen by studying the simplified example shown in figure 8-26. Let AB be a 100-foot chord of the curve. Assume that you decide to set one more point, C, on the



45.357
Figure 8-26.–Subchords by chord definition.

middle of the curve. The two chords AC and CB are, nominally, "50-foot chords", but, since there are two equal sides opposite the 100-foot hypotenuse in triangle ABC, they obviously have to be longer than 50 ft.

Methods which may be used to determine actual subchord length include (1) the use of a formula, (2) the use of tables of excess of arc and ratio of correction, and (3) the use of a table of corrections for subchord lengths.

Let c be the actual length of a subchord. Let d be the central angle subtended by c. As shown before (see fig. 8-7):

$$\sin\frac{D}{2} = \frac{50}{R}$$

Therefore $$\sin\frac{D}{2} = \frac{100}{2R}$$

Similarly, $$\sin\frac{d}{2} = \frac{c}{2R}$$

Since they are proportional,

$$\frac{\sin\frac{D}{2}}{\sin\frac{d}{2}} = \frac{\frac{100}{2R}}{\frac{c}{2R}}$$

Cross multiplying and solving for c:

$$\left(\sin\frac{D}{2}\right)\left(\frac{c}{2R}\right) = \left(\sin\frac{d}{2}\right)\left(\frac{100}{2R}\right)$$

$$\left(\sin\frac{D}{2}\right)(c) = \left(\sin\frac{d}{2}\right)(100)$$

$$c = 100\,\frac{\sin\frac{d}{2}}{\sin\frac{D}{2}}$$

This last expression is a formula you may use to compute the actual length of c.

The actual arc length is always greater than the chord; the amount of excess increases as the curvature (D) increases. Figure 8-27 shows a sample page from a table of long chords and actual arcs. As you can see from the table, the excess of arc for one station is 0.127 ft for a 10° curve and 0.326 ft for a 16° curve. Now let us call the amount by which an actual subchord length exceeds the nominal length the "correction" of the subchord. This correction for any subchord bears an almost constant ratio to the excess of arc per station, whatever the degree of curvature might be. Figure 8-28 shows this ratio for various nominal subchords. To determine the actual subchord length in a given case, multiply the excess of arc per station for the given degree of curve (fig. 8-27) times the ratio of correction (fig. 8-28) for the given subchord nominal length.

The third method of determining the actual subchord length is to interpolate from a table of corrections for subchord lengths. Figure 8-29 shows such a table, together with an explanation and an example.

Now apply these methods of computing subchords to the curve shown in figure 8-22. Set up a curve stakeout table (fig. 8-30) similar to the one prepared for the arc definition curve. As before, the first column shows the stations. The second column is headed "length" instead of "arc" in this case, because the length of the curve is measured along the 100-foot chord lengths and the nominal subchord lengths. The third, fourth, and fifth columns, showing the Δ, the deflection angle, and the total deflection angle, are computed in exactly the same manner as for the arc definition example. The first chord length in the last column is the actual length of the nominal 46.17-foot chord. You may use the following formula to compute it:

$$c = 100\,\frac{\sin\frac{d}{2}}{\sin\frac{D}{2}} = 100\,\frac{\sin 2^\circ 18'30''}{\sin 5^\circ 00'00''}$$

$$= 100\,\frac{.040277}{.087156} = 46.21$$

Degree of curve		Actual arc, 1 station	Long chords							Degree of curve	
			2 sta.	3 sta.	4 sta.	5 sta.	6 sta.	7 sta.	8 sta.	°	′
10	0	100.127	199.2	297.0	392.4	484.9	573.7	658.1	737.5	10	0
	10	131	99.2	96.9	92.2	84.4	72.8	56.7	35.5		10
	20	136	99.2	96.8	91.9	83.9	71.9	55.3	33.4		20
	30	140	99.2	96.7	91.7	83.4	71.0	53.9	31.3		30
	40	145	99.1	96.5	91.4	82.8	70.1	52.4	29.1		40
	50	149	99.1	96.4	91.1	82.3	69.2	51.0	27.0		50
11	0	100.154	199.1	296.3	390.8	481.8	568.2	649.5	724.8	11	0
	10	158	99.1	96.2	90.6	81.2	67.3	48.0	22.5		10
	20	163	99.0	96.1	90.3	80.7	66.3	46.5	20.3		20
	30	168	99.0	96.0	90.0	80.1	65.3	44.9	18.0		30
	40	173	99.0	95.9	89.7	79.5	64.3	43.3	15.7		40
	50	178	98.9	95.7	89.4	78.9	63.3	41.8	13.3		50
12	0	100.183	198.9	295.6	389.1	478.3	562.3	640.1	710.9	12	0
	10	188	98.9	95.5	88.8	77.7	61.3	38.5	08.5		10
	20	193	98.8	95.4	88.5	77.1	60.2	36.9	06.1		20
	30	199	98.8	95.3	88.2	76.5	59.2	35.2	03.7		30
	40	204	98.8	95.1	87.9	75.9	58.1	33.5	01.2		40
	50	209	98.7	95.0	87.6	75.3	57.0	31.8	698.6		50
13	0	100.215	198.7	294.9	387.2	474.6	555.9	630.1	696.1	13	0
	10	220	98.7	94.7	86.9	74.0	54.8	28.3	93.5		10
	20	226	98.6	94.6	86.6	73.3	53.7	26.5	90.9		20
	30	232	98.6	94.5	86.3	72.7	52.5	24.8	88.3		30
	40	237	98.6	94.3	85.9	72.0	51.4	22.9	85.7		40
	50	243	98.5	94.2	85.6	71.3	50.2	21.1	83.0		50
14	0	100.249	198.5	294.1	385.2	470.6	549.1	619.3	680.3	14	0
	10	255	98.5	93.8	84.9	70.0	47.9	17.4	77.5		10
	20	261	98.4	93.8	84.5	69.3	46.7	15.5	74.8		20
	30	267	98.4	93.6	84.2	68.6	45.5	13.6	72.0		30
	40	274	98.4	93.5	83.8	67.8	44.2	11.7	69.2		40
	50	280	98.3	93.3	83.4	67.1	43.0	09.8	66.3		50
15	0	100.286	198.3	293.2	383.1	466.4	541.7	607.8	663.5	15	0
	10	293	98.3	93.0	82.7	65.7	40.5	05.8	60.6		10
	20	299	98.2	92.9	82.3	64.9	39.2	03.8	57.7		20
	30	306	98.2	92.7	81.9	64.2	37.9	01.8	54.8		30
	40	312	98.1	92.6	81.5	63.4	36.6	599.8	51.8		40
	50	319	98.1	92.4	81.2	62.6	35.3	97.8	48.8		50
16	0	100.326	198.1	292.3	380.8	461.9	534.0	595.7	645.8	16	0
	10	332	98.0	92.1	80.4	61.1	32.6	93.6	42.8		10
	20	339	98.0	91.9	80.0	60.3	31.3	91.5	39.7		20
	30	346	97.9	91.8	79.6	59.5	29.9	89.4	36.6		30
	40	353	97.9	91.6	79.1	58.7	28.6	87.3	33.6		40
	50	361	97.8	91.4	78.7	57.9	27.2	85.1	30.4		50
Degree		Actual arc	2 sta.	3 sta.	4 sta.	5 sta.	6 sta.	7 sta.	8 sta.	Degree	

45.45.9

Figure 8-27.—Sample page from table of long chords and actual arcs (chord definition).

Nominal Length of Subchord	Ratio	Nominal Length of Subchord	Ratio
0	0.000	55	0.383
5	0.050	60	0.383
10	0.099	65	0.374
15	0.147	70	0.356
20	0.192	75	0.327
25	0.234	80	0.287
30	0.273	85	0.235
35	0.307	90	0.169
40	0.336	95	0.092
46	0.358	100	0.000
50	0.374		

45.45.10

Figure 8-28.—Table of ratio of correction of subchords to the excess of arc per station (chord definition).

Or, you may determine the excess per station (0.127) for a 10° curve from figure 8-27; then multiply the excess by the ratio of correction (0.36) interpolated for 46.17 feet from figure 8-28. The correction to be added to 46.17, then, is (0.127) × (0.36) or 0.046, which you may call 0.05. The actual chord, then, computes to be 46.17 plus 0.05 or 46.22 ft. Or, to apply the third method, you may interpolate in figure 8-29 opposite 10° between 25 ft and 50 ft, and determine a correction of 0.045 (which you may call either 0.04 or 0.05) to be added to 46.17, giving you either 46.21 or 46.22. Note that all methods provide answers within a few thousandths of a foot of one another; such differences are not significant for most fieldwork. If a more exact value were needed, however, you could use a more precise value of d/2 in the equation and get your answer to the thousandth. The remaining chords in the last column of figure 8-30 require no correction; the 100-foot chords are exact because you are using the chord definition of degree of curvature, and, as before, the last chord is small enough to need no correction.

Intermediate Set-up

In the foregoing examples, the assumption was made that the entire curve could be staked out with the instrument set up on the P.C. This is not always possible; there may be obstructions on the line of sight, or the curve may be so long that it isn't practical to keep the instrument at the P.C. In these cases, it is necessary to move ahead on the curve and set up at an intermediate point on the curve. Assume that, in figure 8-22 station 80 + 00 has been staked from the P.C., that obstructions prevent your sighting station 81 + 00, and that you must move ahead to station 80 + 00.

To understand the method of training the telescope from one station to a subsequent station, study figure 8-31. This figure shows a semicircular curve, running from A to D, and a tangent at A. The curve is divided into three intervals of arc, and the stations are connected by three equal chords: AB, BC, and CD. The chord from A to B is chord AB; chords from A to C and from A to D are indicated by the dotted lines AC and AD.

The deflection angle between AB and the tangent is 30°; this is the COMPUTED DEFLECTION ANGLE for station B. Then the computed deflection angle for station C is 2 × 30°, or 60°; and the computed deflection angle for station D is 3 × 30°, or 90°. Remember that the computed deflection angle for any station is the angle between the tangent at the P.C. and the chord running from the P.C. to the station.

The deflection angle between chord BC and chord AB (extended) measures TWICE the size of the deflection angle between chord AB and the tangent. If you were set up at station B, you could turn this deflection angle by plunging the telescope, sighting back on A, setting the vernier at 0, replunging, and turning 60° to the right. That is, you could turn the deflection angle from station B to station C by backsighting on A, setting the computed deflection angle of station A (which is 0°) on the vernier, and then turning (to the right, in this case) an angle equal in size to the computed deflection angle for station C.

Similarly, if you were set up at station C, you could train the telescope on station D as follows: Backsight on station B, and set the computed deflection angle of station B (30°) on the vernier. Then plunge the telescope, and turn right until you read the computed deflection angle for station D (90°) on the vernier. Or, backsight on station A and set the computed

CORRECTIONS FOR SUBCHORD LENGTHS

(CHORD DEFINITION)

For setting stakes for sharp curves it is often necessary to use chord lengths less than 100 feet. The most common chord lengths are 50, 25, 20, and 10 feet, in order as the degree of curvature increases. Common and accepted practice prescribes that the length of a curve be measured in chords 100 feet long and fractions thereof. The arc of a curve corresponding to a 100-foot chord is greater than 100 feet. Similarly the sum of 2, 4, 5, or 10 equal subchords which subtend the arc measured by a 100-foot chord is greater than 100 feet. The nominal lengths of these subchords are 50, 25, 20, or 10 feet, respectively. To these nominal lengths add the corrections shown in the table to give the actual lengths of the subchords. *Example:* Suppose it is desirable to set stakes by subchords of 25-foot nominal length on a 20° curve. Enter the table for $D = 20°$. On this line, in the column headed 25, is found the correction .119. To 25 add .119. This gives 25.119 feet as the actual length of the subchord whose nominal length is 25 feet.

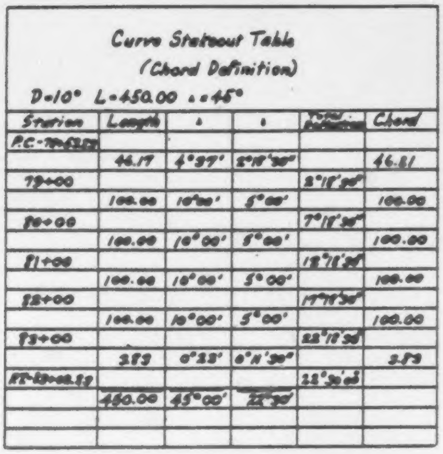
D.	50′	25′	20′	10′	D.	50′	25′	20′	10′
°					°				
3	.004	.003	.002	.001	27	.349	.218	.179	.092
4	.008	.005	.004	.002	28	.376	.235	.192	.099
5	.012	.008	.006	.003	29	.403	.252	.206	.106
6	.017	.011	.009	.005	30	.431	.270	.221	.114
7	.023	.015	.012	.006	31	.461	.288	.236	.122
8	.030	.019	.016	.008	32	.491	.307	.252	.130
9	.039	.024	.020	.010	33	.523	.327	.268	.138
10	.048	.030	.024	.013	34	.555	.347	.285	.147
11	.058	.036	.030	.015	35	.589	.368	.302	.156
12	.069	.043	.035	.018	36	.623	.390	.320	.165
13	.081	.050	.041	.021	37	.659	.412	.338	.174
14	.093	.058	.048	.025	38	.695	.435	.356	.184
15	.107	.067	.055	.028	39	.733	.458	.376	.194
16	.122	.076	.063	.032	40	.771	.483	.396	.204
17	.138	.086	.071	.036	41	.811	.507	.416	.214
18	.155	.097	.079	.041	42	.851	.533	.437	.225
19	.172	.108	.088	.045	43	.893	.559	.458	.236
20	.191	.119	.098	.050	44	.936	.586	.480	.248
21	.211	.132	.108	.056	45	.980	.613	.502	.259
22	.231	.144	.118	.061	46	1.024	.641	.525	.271
23	.253	.158	.130	.067	47	1.070	.670	.549	.283
24	.275	.172	.141	.073	48	1.117	.699	.573	.296
25	.299	.187	.153	.079	49	1.165	.729	.598	.308
26	.323	.202	.166	.085	50	1.214	.760	.623	.321

45.45.11

Figure 8-29.—Table of corrections for subchord lengths (chord definition).

deflection angle for station A (0°) on the vernier. Then plunge the telescope and turn right until you read the computed deflection angle for station D (90°) on the vernier.

Suppose you wanted to shoot station D from station B. The chord from station B to station D is the dotted line BD. You can see that the deflection angle between chord AB and this chord measures 90°, and you can see that the same rule applies. Backsight on A, and set the computed deflection angle for A (0°) on the vernier. Then plunge the telescope, and turn until you read the computed deflection angle for D (90°) on the vernier.



Curve Stakeout Table
(Chord Definition)
D=10° L=450.00 Δ=45°

Station	Length	Δ	δ	Total Deflection	Chord
P.C.-78+53.83					
	46.17	4°37'	2°18'30"		46.81
79+00				2°18'30"	
	100.00	10°00'	5°00'		100.00
80+00				7°18'30"	
	100.00	10°00'	5°00'		100.00
81+00				12°18'30"	
	100.00	10°00'	5°00'		100.00
82+00				17°18'30"	
	100.00	10°00'	5°00'		100.00
83+00				22°18'30"	
	3.83	0°23'	0°11'30"		3.83
P.T.-83+03.83				22°30'00"	
	450.00	45°00'	22°30'		

45.358

Figure 8-30.—Curve stakeout table (chord definition).

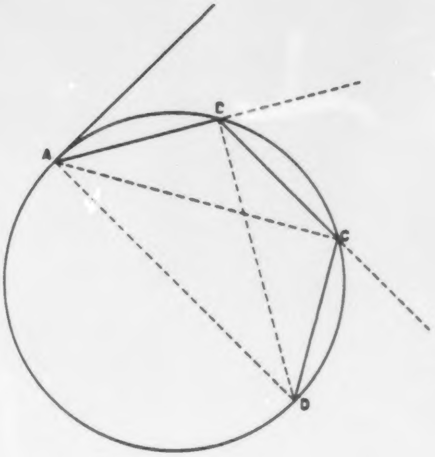


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Figure 8-31.—Deflection angles on a semicircle.

The rule, then, for turning the deflection angle between any two adjoining chords is this:

1. Plunge the telescope, backsight on the appropriate station, and set the vernier to read the computed deflection angle of that station.
2. Replung the telescope, and turn until you read the computed deflection angle of the other appropriate station.

Let's go back, now, and compute the deflection angles for the curve (arc definition) shown in figure 8-22. The computed deflection angle for station 79 + 00 is 2°17'30"; you have already turned that one. The computed deflection angle for station 80 + 00 is 7°17'30". These and the deflection angles for the other stations on the arc are shown in figure 8-25.

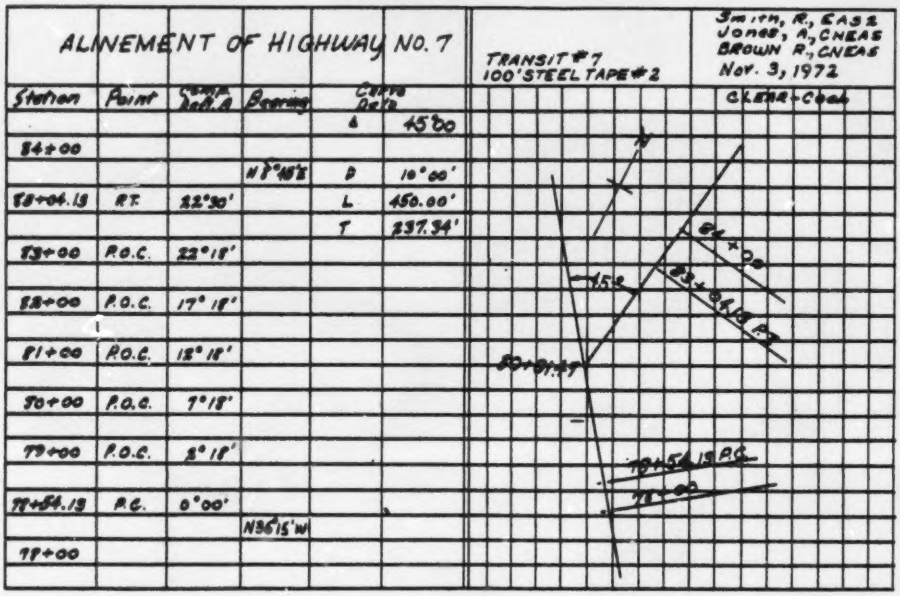
You are set up at station 80 + 00. To turn the deflection angle between the chord from the P.C. to 80 + 00 and the chord from 80 + 00 to 81 + 00, backsight on the P.C. and set the vernier to read the computed deflection angle of that station, which is 0°. Then plunge the telescope, and turn until you read the computed deflection angle of station 81 + 00 (which is 12°17'30") on the vernier. When you have turned the angle, measure off a full chord (99.87 ft) from station 80 + 00 along the line of sight. This locates station 81 + 00.

You can go ahead and locate station 82 + 00 with the transit still at station 80 + 00, or you can shift to station 81 + 00 if field conditions so demand. If you stay at station 80 + 00, you locate station 82 + 00 as follows: turn right until you read the computed deflection angle of station 82 + 00 (17°17'30") on the vernier. When you have turned the angle, measure off one full chord (99.87 ft) from station 81 + 00 to locate station 82 + 00. Follow the same procedure to set 83 + 00 and to check in to the P.T.

In addition to being staked out from the P.C. and from intermediate stations, a curve may be staked out with the instrument set up at the P.T. If you want the chainmen to chain ahead from the P.C. to the P.T., set the circle at 0°, back sight on the P.C., and lay off the same deflection angles you would use if the transit were at the P.C. If you want the chainmen to chain back from the P.T. to the P.C., set the circle at the total deflection (Δ/2), back sight on the P.I., and turn in succession (toward the P.C.) to each deflection angle (computed for each station) from the P.T. back to the P.C.; when you make the last setting, your plate should read 0° and you should be sighting the P.C.

Field Notes for Curve Layout

Figure 8-32 shows field notes for the curve you staked out in figure 8-22. On the remarks



ALINEMENT OF HIGHWAY NO. 7

TRANSIT #7
100' STEEL TAPE #2

Smith, R., EAS2
Jones, A., CNEAS
Brown R., CNEAS
Nov. 3, 1972

CLEAR-Cool

Station	Point	Comp. Defl. A	Bearing	Curve Data	
				Δ	45°00
84+00					
			N8°45'E	D	10°00'
83+04.19	P.T.	22°30'		L	450.00'
				T	237.34'
83+00	P.O.C.	22°18'			
82+00	P.O.C.	17° 18'			
81+00	P.O.C.	12° 18'			
80+00	P.O.C.	7°18'			
79+00	P.O.C.	2° 18'			
78+54.19	P.C.	0°00'			
			N36°15'W		
78+00					

N
84+00
83+04.19 P.T.
45°
80+91.47
78+54.19 P.C.
78+00

45.7.11

Figure 8-32.—Field notes.

page, make a sketch, to a suitable scale, of the tangents, and locate and mark such points as the last full station before the P.C., the P.C., the P.I., the P.T., and the first full station after the P.T.

In the two right-hand columns of the data page, set down the curve data, such as the central angle (Δ), the degree of curvature (D), the length of the curve (L), and the tangent (T).

In the first column on the data page, under "Station," list the stations–not only those which appear in the sketch, but also those full stations which lie on the curve. Since the sketch reads from the bottom up (so that data will be presented as it appears in the field as you look ahead on line), you list your stations from the bottom up on the page.

In the second column on the data page, under "Point," list the character of each station–that is, whether it is the P.C., the P.T., a P.O.L. (point on line, such as stations 78 + 00 and 84 + 00), or a P.O.C. (point on curve, such as the stations from 79 + 00 to 83 + 00 inclusive).

Next to each station on the curve (including the P.C. and the P.T.), in the column headed "Comp. Defl. A.," you list the computed deflection angle (that is, the angle between the tangent through the P.C. and the chord from the P.C. to the station) for that particular station. The computed deflection angle for the P.C. is 0°. The computed deflection angle for the P.T. is one-half of the central angle. You have computed the angles for the other stations.

Offset Curves

OFFSET CURVES are run outside a construction when it is not possible to run the centerline, or when an accurate alignment control is needed for the actual limits of the borders of a construction. In the case of a pavement, for example, offset curves are usually run on either side of the centerline and at the same distance from it.

Figure 8-33 shows a possible offset layout. Usually offset stations are set and marked to agree with the stationing along the centerline. Note that the radius to any point on a circular curve is perpendicular to the tangent which it supports along the centerline. Therefore, all the

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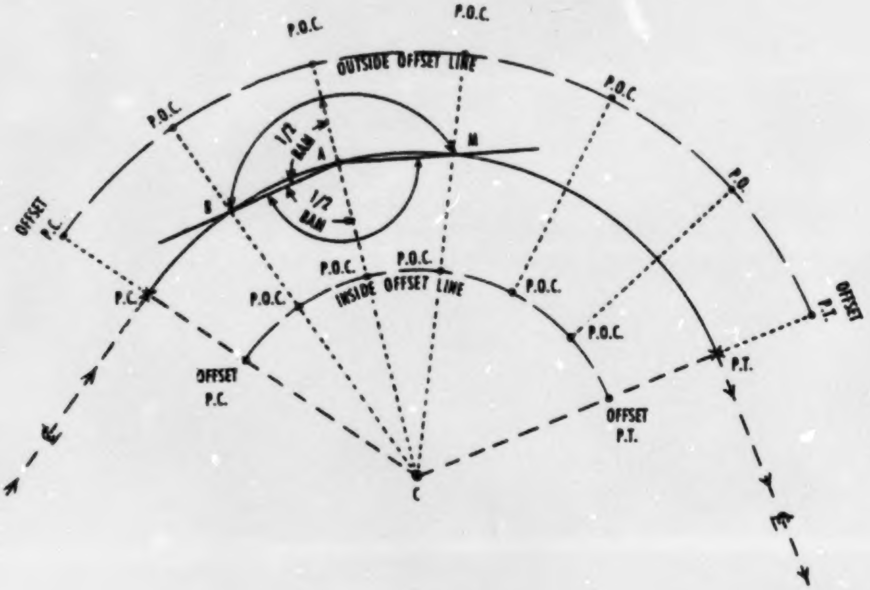

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Figure 8-33.–Offset layout.

P.O.C.'s on the offset curves will be set at right angles to the supporting tangents.

If the centerline is already in, and equal chords have been used, P.O.C.'s on the offset curves can be located ~~by~~ bisecting the complete angle between two adjacent P.O.C.'s. If, for example, you are set up at station A in figure 8-33, bisect angle BAM using either the outside or the inside angle. Now, with the line of sight determined, the offset station can be measured out from the centerline station to the specified offset distance.

When an offset curve is laid out without the centerline curve, compute the curve elements using the same Δ and an R increased or reduced by the distance from the centerline to the offset.

COMPOUND AND REVERSED CURVES

A COMPOUND CURVE consists of two or more simple curves of different degrees of curve following one another in the same general direction and having a common tangent at the point where they join. The point where the two curves run together is called a POINT OF COMPOUND CURVE (P.C.C.).

The following is a practical application of a compound curve layout. When running a preliminary road survey through mountainous terrain, you will most likely select a course that offers least resistance to construction. Instead of tunneling through a steep hill, follow the side of the hill, increasing or decreasing the elevation to correspond with the connecting elevation on the opposite side. In following around a hill, it will be necessary to lay out shorter tangent lines than usual with a different P.I. at each change in direction. To change over from the tangent lines to the actual proposed centerline of the road; it may be feasible to have a curved section similar to that shown in figure 8-34.

SB is the radius of curve No. 1 and SA is the radius of curve No. 2. In laying out a compound curve to intercept the course running into curve No. 1 and out from curve No. 2, it is necessary, for true alignment, that the sum of the tangents to curve No. 1 and curve No. 2 be equal to the length of the common tangent.

Begin as you would in laying out a simple circular curve from P.C. to the point of compound curvature P.C.C. Set up the transit at the P.C.C. Set the circle at half the central angle of

670

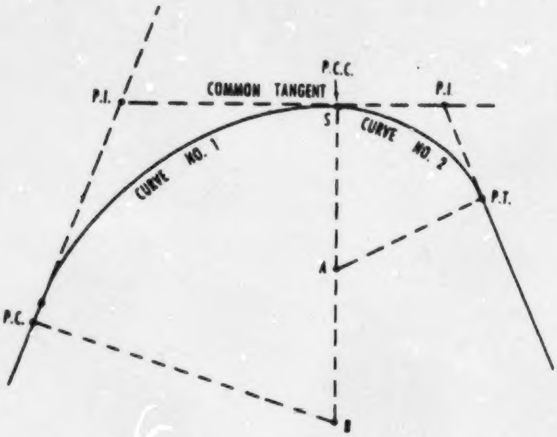

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Figure 8-34.—Compound curve.

curve No. 1 and backsight on the P.C. If you now turn the circle to 0°, you should be sighting along the common tangent at the P.C.C. Lay out curve No. 2 just like any other simple curve, using the deflection angle method.

A reversed curve consists of two simple curves which turn in opposite directions as shown in figure 8-35. The two simple curves have a common tangent, where they join, but their centers lie on opposite sides of the common tangent. The point where the two curves join is called A POINT OF REVERSED CURVE (P.R.C.). The two curves may be of the same or different degrees of curve. Figure 8-35 shows a double reversed curve of two different degrees of curve. Curve ABC is a single reversed curve with the same degree of curve. Curve BCD shows two different degrees of curve. Reversed curves are used to advantage in the location of railroad cross-overs and spur tracks leading to manufacturing or industrial plants. Lay out the first section of a reversed curve just like any other simple circular curve. Set up the transit at the point of reversed curve (P.R.C.). Adjust the circle so that it sights 0° along the common tangent. Lay out the second simple curve on the opposite side of the common tangent. The P.R.C. at B may be considered as the P.T. of curve No. 1 and also as the P.C. of curve No. 2. Set the instrument up at the second P.R.C. at C and repeat the layout procedure.

The principles of computation used for simple curves apply to all horizontal curves. Compound and reversed curves merely continue on from the

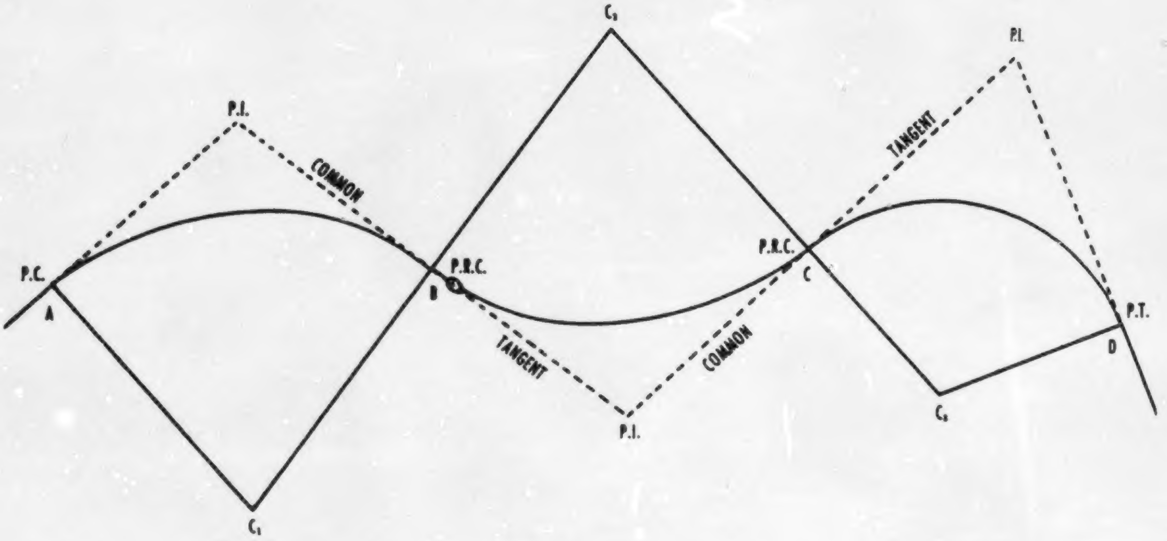

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Figure 8-35.—Reversed curves.

end of the preceding curve. Like any simple curve, a compound or reversed curve finally ends in a P.T.

SPIRAL TRANSITION CURVES

SPIRAL CURVES are transition curves used in railroad and highway work to provide a gradual change between a straight and a curved part of the road. Transition curves provide safer and more comfortable riding by reducing the tendency of the vehicle to lurch and to skid while going around an abrupt curve in the roadway. A more gradual transition from a straight to a curved section of roadway also gives the motorist more time and a greater distance within which to adjust his steering wheel to the anticipated curve. Spirals, therefore, should be as long as possible. They may be true spirals or a consecutive series of compound curves. The degree of curve of a spiral changes uniformly with the distance from some point of reference. For example, note in figure 8-36 that the spiral is relatively flat at the T.S. (the point of change from tangent to spiral). This point represents the beginning of the spiral where the curve starts diverging from the main tangent. As the spiral continues from the T.S. to the S.C. (the point of change from the spiral to circle), the spiral gradually becomes sharper until its radius becomes equal to that of the circular curve to which it is connected.

The spiral merges into the circular curve between the S.C. (point of change from spiral to circle) and the C.S. (point of change from circle to spiral). Approaching the end of the curve, the spiral once again flattens out between the C.S. and the S.T. (point of change from spiral to tangent). Figure 8-36 shows three distinct sections of the total curve, the central portion being the circular curve, merging gradually at each end into a spiral transition curve.

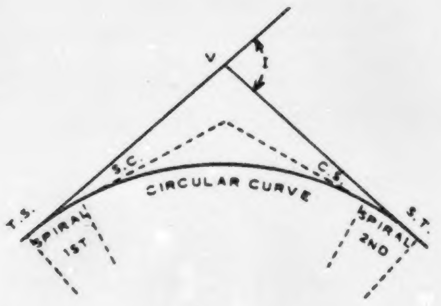

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Figure 8-36.—Spiral transitions.

A spiral curve is laid out by setting up the instrument at the T.S. and laying out the first spiral to the S.C. using chords and deflection angles measured from the main tangent. Tables are available from which you can obtain deflection angles (θ) of a spiral curve from the T.S. to any point on the spiral. For most highway work it is conventional to use the 10-point spiral. It may increase your understanding to compare the layout of a spiral with that of a circular curve. The T.S. of a spiral may be compared to the P.C. of a circular curve; the S.C. may be compared to the P.T. of a circular curve. After laying out the first spiral, move the transit to the S.C. and lay out the circular to its end at the C.S., using chords and deflection angles measured from an auxiliary tangent at the S.C. Move the transit to the S.T. and lay out the second spiral, working back toward the C.S. The end of the second spiral should coincide with the C.S. and thereby provide a check on the accuracy of your work.

SUPERELEVATION AND WIDENING

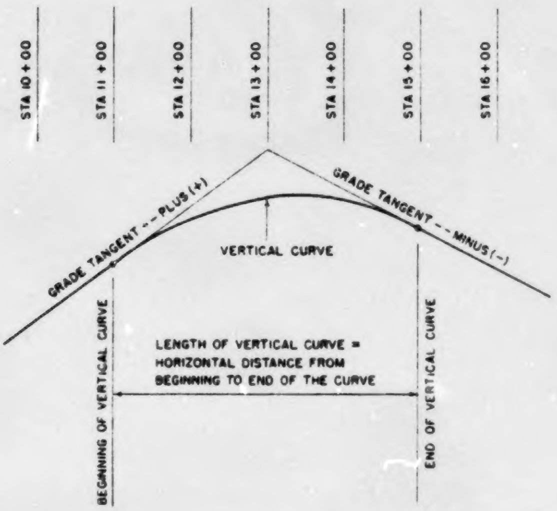
"Superelevation" is the term used by the engineer to describe the banking of a curved roadway. Superelevations are closely associated with spiral transition curves in that they both attempt to compensate for the centrifugal force which tends to cause skidding outward from the center of a curve. Straight sections of highways are usually built with a slight convexity or crown to take care of drainage. If the same crown were continued around curves, it would produce a hazard for fast moving traffic. To minimize the danger of skidding, engineers superelevate or bank the outer portion of the curved section of the road. At the same time, they usually widen the pavement as an additional safety measure. No attempt will be made to discuss the computations involved in transition spirals, in superelevations, or in road widening, because they are beyond the scope of this book. Such information is available in standard textbooks on highway engineering.

VERTICAL CURVES

In addition to horizontal curves which go to the right or left, roads also have curves that go up or down. These curves in a vertical plane are called VERTICAL CURVES. Vertical curves at a crest or the top of a hill are called SUMMIT CURVES or OVERVERTICALS. Vertical curves at the bottom of a hill or dip are called SAG CURVES or UNDERVERTICALS.

GRADES

Vertical curves are used to connect stretches of road which go up or down at a constant slope. These lines of constant slope are called GRADE TANGENTS (see fig. 8-37.) The rate of slope is called the GRADIENT, or simply the GRADE. (Do not confuse this use of the term "grade" with other meanings such as the design elevation of a finished surface at a given point, or the actual elevation of the existing ground at a given point.) Grades which ascend in the direction of the stationing are designated "plus"; those which descend in the direction of the stationing are designated "minus." Grades are measured in terms of percent; that is, the number of feet of rise or fall in a 100-foot horizontal stretch of the road.


STA 10+00
STA 11+00
STA 12+00
STA 13+00
STA 14+00
STA 15+00
STA 16+00
GRADE TANGENT -- PLUS (+)
GRADE TANGENT -- MINUS (-)
VERTICAL CURVE
LENGTH OF VERTICAL CURVE = HORIZONTAL DISTANCE FROM BEGINNING TO END OF THE CURVE
BEGINNING OF VERTICAL CURVE
END OF VERTICAL CURVE

45.365

Figure 8-37.—A vertical curve.

After the location of a road has been determined and the necessary fieldwork has been obtained, the engineer designs, or "fixes", or "sets", the grades. A number of factors are considered, including the intended use and importance of the road, and the existing topography. If a road is too steep, the comfort and safety of the users and fuel consumption of the vehicles will be adversely affected. Therefore, the design criteria will specify MAXIMUM GRADES. Typical maximum grades are 4 percent desired maximum, and 6 percent absolute maximum, for a primary road. (The 6 percent means, as indicated before, a 6-foot rise for each 100 feet ahead on the road.) For a secondary road or major street, the maximum grades might be 5 percent desired and 8 percent absolute maximum; and for a tertiary road or a secondary street, 8 percent desired and 10 percent (or perhaps 12 percent) absolute maximum. Conditions may sometimes demand that grades or ramps, driveways, or short access streets go as high as 20 percent. The engineer must also consider MINIMUM GRADES. A street with curb and gutter must have enough fall so that the storm water will drain to the inlets; 0.5 percent is a typical minimum grade for curb and gutter (that is, 1/2 foot, or 6 inches. minimum fall for each 100 feet ahead). For roads with side ditches, the desired minimum grade might be 1 percent; but since ditches may slope at a grade different from the pavement, a road may be designed with a 0 percent grade. Zero percent grades are not unusual, particularly through plains or tidewater areas. Another factor that is considered in designing the finished profile of a road is the EARTHWORK BALANCE. That is, the grades should be set so that all the earth cut off the hills may be economically hauled to fill in the low areas. In the design of urban streets, the best utilization of the building sites adjacent to the street will generally take precedence over seeking an earthwork balance.

COMPUTING VERTICAL CURVES

As you have previously learned, the horizontal curves used in highway work are generally the arcs of circles. But vertical curves are usually PARABOLIC CURVES. The parabola was chosen primarily because its shape provides a transition, and because it lends itself to computational procedures which are described in the next section of this chapter. Designing a vertical curve consists principally of deciding on the appropriate LENGTH of the curve. As indicated in figure 8-37, the length of a vertical curve is the HORIZONTAL DISTANCE from the beginning to the end of the curve; the length of the curve is NOT the distance along the parabola itself. The longer a curve is, the more gradual will be the transition from one grade to the next; the shorter the curve, the more abrupt the change. The change must be gradual enough to provide the required SIGHT DISTANCE (see figure 8-38). The sight distance requirement will depend on the speed for which the road is designed, whether passing or nonpassing distance is required, and other assumptions such as reaction time, braking time, stopping distance, height of eye, height of object, and so on. Typical heights of eye used are 4.5 feet or, more recently, 3.75 feet; typical heights of object are 4 inches to 1.5 feet. For a sag curve, the sight distance will usually not be significant during daylight; but consideration must be given to nighttime when the reach of headlights may be limited by the abruptness of the curve. (See fig. 8-38.)

Elements of Vertical Curves

Figure 8-39 shows the elements of a vertical curve. The meaning of the symbols and the units of measurement usually assigned to them follow:

P.V.C.	point of vertical curvature; the place where the curve begins.
P.V.I.	point of vertical intersection; where the grade tangents intersect.
P.V.T.	point of vertical tangency; where the curve ends.
P.O.V.T.	point on vertical tangent, applies to an infinite number of points on either tangent.
P.O.V.C.	point on vertical curve; applies to any point on the parabola.
g_1	grade of the tangent on which the P.V.C. is located; measured in percent of slope.
g_2	grade of the tangent on which the P.V.T. is located; measured in percent of slope.
G	the ALGEBRAIC DIFFERENCE of the grades; $G = g_2 - g_1$ wherein plus values are assigned to uphill grades and minus values to downhill grades; example

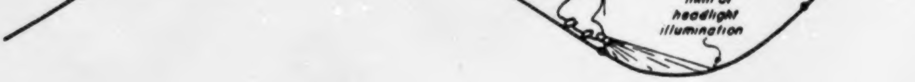


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Figure 8-38.—Sight distance.

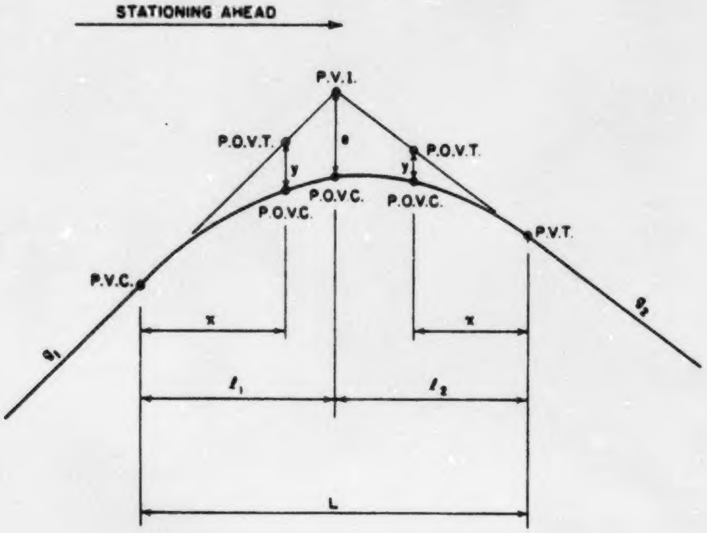

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Figure 8-39.—Elements of a vertical curve.

	of various algebraic differences are shown later in this section.
L	length of the curve; the HORIZONTAL length in 100-foot stations from the P.V.C. to the P.V.T.
l_1	horizontal length of the portion of the P.V.C. to the P.V.I.; measured in feet.
l_2	horizontal length of the portion of the curve from the P.V.I. to the P.V.T.; measured in feet.
e	vertical (external) distance from the P.V.I. to the curve, measured in feet; $e = \frac{LG}{8}$, where L is the total length in stations and G is the algebraic difference of the grades in percent.
x	horizontal distance from the P.V.C. to any P.O.V.C. or P.O.V.T. back of the P.V.I., or the distance from the P.V.T. to any P.O.V.C. or P.O.V.T. ahead of the P.V.I., measured in feet.
y	vertical distance (offset) from any P.O.V.T. to the corresponding P.O.V.C., measured in feet; $y = (x/l)^2(e)$, which is the fundamental relationship of the parabola that permits convenient calculation of the vertical offsets.

The vertical curve computation takes place after the grades have been set and the curve designed. Therefore, at the beginning of the detailed computations, the following are known: g_1, g_2, l_1, l_2, L, and the elevation of the P.V.I. The general procedure is to compute the elevations of certain P.O.V.T.'s; then, using the foregoing formulas, to compute G, thence e, and thence the y's, that correspond to the selected P.O.V.T.'s. Adding or subtracting the y from the elevation of the P.O.V.T. gives the elevation of the P.O.V.C.; that is, the finished elevation on the road, which is the end result being sought. In figure 8-39, the y is subtracted from the elevation of the P.O.V.T. to get the elevation on the curve; but in the case of a sag curve the y is added to the P.O.V.T. elevation to obtain the P.O.V.C. elevation.

The computation of G requires careful attention to the signs of g_1 and g_2. Vertical curves are used at changes of grade other than at the top or bottom of a hill; for example, an uphill grade which intersects an even steeper uphill grade will be eased by a vertical curve. The six possible combinations of plus and minus grades, together with sample computations of G, are shown in figure 8-40. Note that the algebraic sign of G indicates whether to add or subtract y from a P.O.V.T.

The selection of the points at which to compute the y and the elevations of the P.O.V.T. and P.O.V.C. is generally based on the stationing. The horizontal alignment of a road is usually staked out on 50-foot or 100-foot stations; it is customary to compute the elevations at these same points so that both horizontal and vertical information for construction will be provided at the same point. The P.V.C., P.V.I., and P.V.T. are usually set at full stations or half stations. In urban work, elevations are sometimes computed (and staked) every 25 feet on vertical curves. The same, or even closer, intervals may be used on complex ramps and interchanges.

The application of the foregoing fundamentals will be presented in the next two sections under symmetrical and unsymmetrical curves.

Symmetrical Vertical Curves

A SYMMETRICAL VERTICAL CURVE is one in which the horizontal distance from the P.V.I. to the P.V.C. is equal to the horizontal distance from the P.V.I. to the P.V.T. In other words, 1_1 equals 1_2.

The solution of a typical problem dealing with a symmetrical vertical curve will be presented step by step. Assume that you know the following data:

g_1 = + 9%
g_2 = – 7%
L = 400.00′, or 4 stations
The station of the P.V.I. = 30+00
The elevation of the P.V.I. = 239.12 feet

The problem is to compute the grade elevation of the curve, to the nearest hundredth of a foot, at each 50-foot station. Figure 8-41 shows the vertical curve to be solved.

STEP 1: Prepare a table as shown in figure 8-42.

Column 1 shows the stations; column 2, the elevation on tangents; column 3, the ratio of x/1; column 4, the ratio of $(x/1)^2$; column 5, the vertical offsets $[(x/1)^2(e)]$; column 6, the grade elevations on the curve; column 7, the first differences; and column 8, the second differences.

STEP 2: Compute the elevations and set the stations on the P.V.C. and the P.V.T.

Knowing the gradients at the P.V.C. and P.V.T., the elevation and station at the P.V.I., you can compute the elevations and set the stations on the P.V.C. and the P.V.T. The gradient (g_1) of the tangent at the P.V.C. is given as + 9%. This means a rise in elevation of 9 feet for every 100 feet of horizontal distance. Since L is 400.00 feet and since this is a symmetrical vertical curve, 1_1 equals 1_2 equals 200.00 feet. Therefore, there will be a difference of 9 x 2 or 18 feet between the elevation at the P.V.I. and the elevation at the P.V.C. The elevation at the P.V.I. in this problem is given as 239.12 feet. The elevation at the P.V.C., therefore, is 239.12 minus 18 or 221.12 feet.

Calculate the elevation at the P.V.T. in a similar manner. The gradient (g_2) of the tangent at the P.V.T. is given as -7%. This means a drop in elevation of 7 feet for every 100 feet of horizontal distance. Since 1_1 equals 1_2 equals 200 feet, there will be a difference of 7 x 2 or 14 feet between the elevation at the P.V.I. and the elevation at the P.V.T. The elevation at the P.V.I. therefore is 239.12 feet minus 14 feet or 225.12 feet.

In setting stations on a vertical curve, remember that the length of the curve (L) is always measured as a horizontal distance. The half-length of the curve is the horizontal distance from the P.V.I. to the P.V.C. In this problem, 1_1 equals 200 feet. This is equivalent to two 100-foot stations and may be expressed as 2 + 00. Thus, if the station at the P.V.C. is 30 + 00 MINUS 2 + 00 or 28 + 00. The station at the P.V.T. is 30 + 00 PLUS 2 + 00 or 32 + 00. List the stations under column 1.

STEP 3: Calculate the elevations at each 50-foot station on the tangent.

From step 2 you know that there is a 9-foot rise in elevation for every 100 feet of horizontal distance from the P.V.C. to the P.V.I. Thus for every 50 feet of horizontal distance there will be

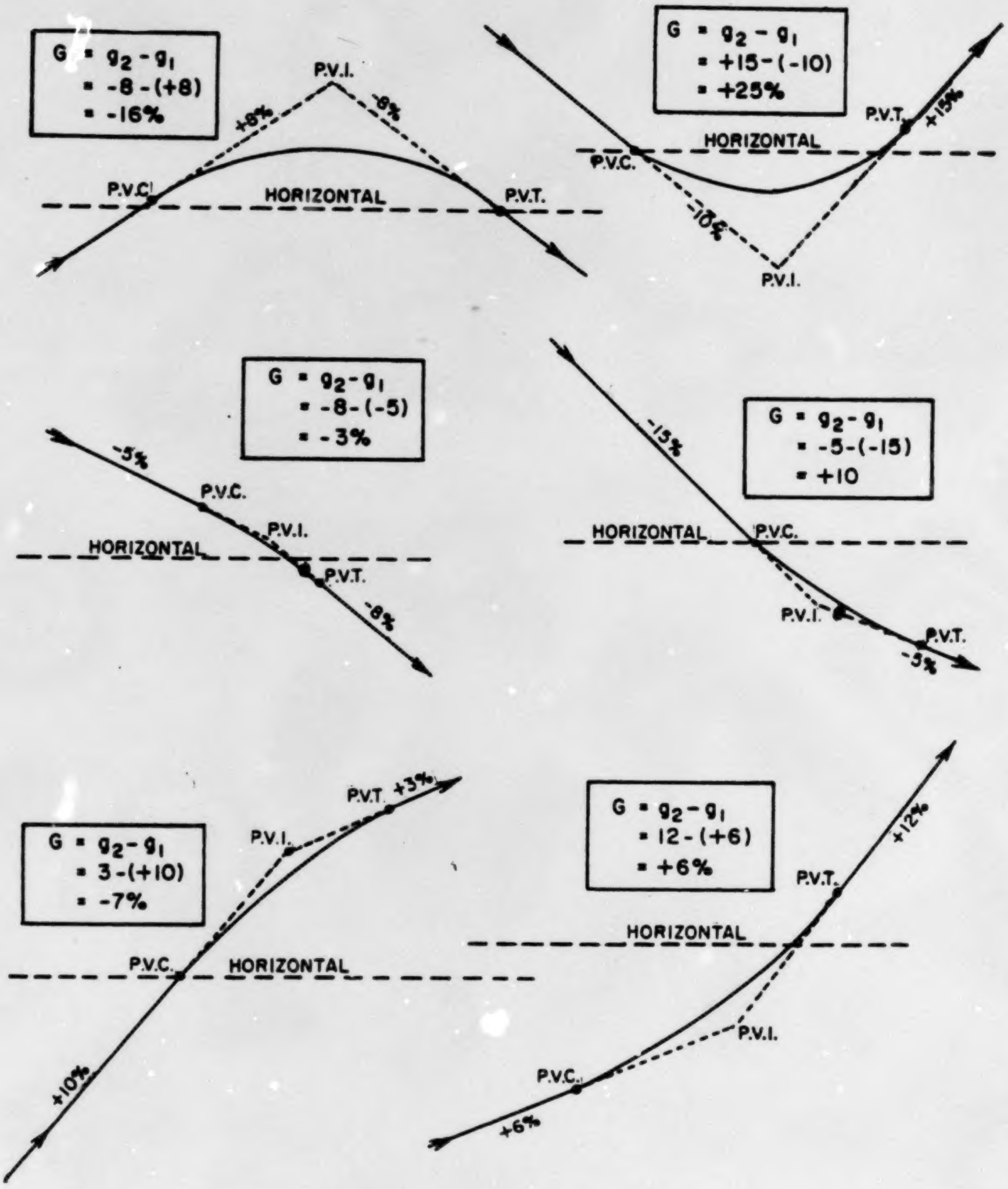

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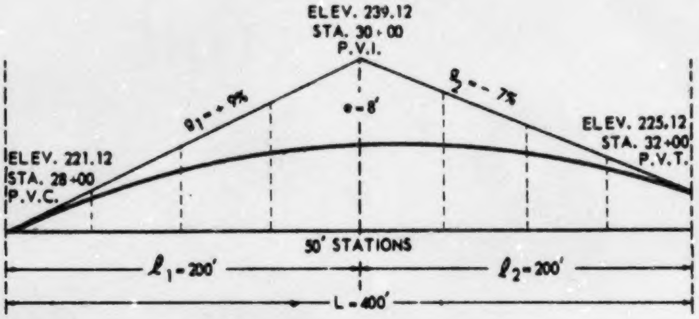
Figure 8-40.—Algebraic differences of grades.



45.369

Figure 8-41.–Symmetrical vertical curve.

a rise of 4.50 feet in elevation. The elevation on the tangent at station 28 + 50 is 221.12 plus 4.50 or 225.62 feet. The elevation on the tangent at station 29 + 00 is 225.62 plus 4.50 or 230.12 feet. The elevation on the tangent at station 29 + 50 is 230.12 plus 4.50 or 234.62 feet. The elevation on the tangent at station 30 + 00 is 234.62 plus 4.50 or 239.12 feet. In this problem, to find the elevation on the tangent at any 50-foot station starting at the P.V.C., add 4.50 to the elevation at the preceding station until you reach the P.V.I. At this point use a slightly different procedure in calculating elevations because the curve slopes downward toward the P.V.T. Think of the elevations as being divided into two groups–one group running from the P.V.C. to the P.V.I.; the other group running from the P.V.T. to the P.V.I.

Proceeding downhill on a gradient of -7% from the P.V.I. to the P.V.T., there will be a drop of 3.50 feet for every 50 feet of horizontal distance. To find the elevations at stations

Stations	Elevations on tangent	x/l	$(x/l)^2$	Vertical offsets	Grade elevation on curve	First difference	Second difference
28+00 (*PVC*)	221.12	0	0	0	221.12		
						+4.00	
+50	225.62	$\frac{1}{4}$	$\frac{1}{16}$	−0.50	225.12		+1.00
						+3.00	
29+00	230.12	$\frac{1}{2}$	$\frac{1}{4}$	−2.00	228.12		+1.00
						+2.00	
+50	234.62	$\frac{3}{4}$	$\frac{9}{16}$	−4.50	230.12		+1.00
						+1.00	
30+00 (*P. V. I.*)	239.12	1	1	−8.00	231.12		+1.00
						.00	
+50	235.62	$\frac{3}{4}$	$\frac{9}{16}$	−4.50	231.12		+1.00
						−1.00	
31+00	232.12	$\frac{1}{2}$	$\frac{1}{4}$	−2.00	230.12		+1.00
						−2.00	
+50	228.62	$\frac{1}{4}$	$\frac{1}{16}$	− .50	228.12		+1.00
						−3.00	
32+00 (*PVT*)	225.12	0	0	0	225.12		

45.370

Figure 8-42.–Table of computations of elevations on a symmetrical vertical curve.

between the P.V.I. and the P.V.T. in this particular problem, SUBTRACT 3.50 from the elevation at the preceding station. The elevation on the tangent at station 30+50 is 239.12 MINUS 3.50 or 235.62 feet. The elevation on the tangent at station 31+50 is 235.62 MINUS 3.50 or 232.12 feet. The elevation on the tangent at station 31+50 is 232.12 MINUS 3.50 or 228.62 feet. The elevation on the tangent at station 32+00 (P.V.T.) is 228.62 MINUS 3.50 or 225.12 feet. The last subtraction provides a check on your work thus far. List the computed elevations under column 2.

STEP 4: Calculate (e), the middle vertical offset at the P.V.I.

First find (G), the algebraic difference of the gradients using the formula

$$G = g_2 - g$$
$$G = -7 - (+9)$$
$$G = -16\%$$

The middle vertical offset (e) is calculated by use of the formula $e = \frac{LG}{8}$, where L is the length of the curve measured in horizontal stations and G is the algebraic difference in gradients.

$$e = \frac{(4)(-16)}{8} = -8.00 \text{ ft}$$

The negative sign indicates e is to be subtracted from the P.V.I.

STEP 5: Compute the vertical offsets at each 50-foot station, using the formula $y = \left(\frac{x}{1}\right)^2 e$.

To find the vertical offset at any point on a vertical curve, first find the ratio x/1, then square it and multiply by e. For example, at station 28+00, the ratio of x/1 = 50/200 = 1/4.

$$\left(\frac{x}{T}\right)^2 = \frac{1}{16}$$

The vertical offset at station 28 + 50 equals (1/16) (−8) or −0.50. Repeat this procedure to find the vertical offset at each of the 50-foot stations. List the results under columns 3, 4, and 5.

STEP 6: Compute the grade elevation at each of the 50-foot stations.

When the curve is on a crest, the sign of the offset will be negative, therefore, subtract the vertical offset (the figure in column 5) from the elevation on the tangent (the figure in column 2). For example, the grade elevation at station 29 + 50 is 225.62 minus 0.50 or 225.12 feet. Obtain the grade elevation at each of the stations in a similar manner. Enter the results under column 6.

NOTE: When the curve is in a dip, the sign will be positive, therefore, you will ADD the vertical offset (the figure in column 5) to the elevation on the tangent (the figure in column 2).

STEP 7. Find the turning point on the vertical curve.

When the curve is on a crest, the turning point is the highest point on the curve. When the curve is in a dip, the turning point is the lowest point on the curve. The turning point will be directly above or below the P.V.I. only when both tangents have the same percent of slope (ignoring the algebraic sign). Otherwise, the turning point will be on the same side of the curve as the tangent with the least percent of slope.

The horizontal location of the turning point is measured from the P.V.C. if the tangent with the lesser slope begins there, or from the P.V.T. if the tangent with the lesser slope ends there. The horizontal location is found by the formula:

$$x_t = \frac{gL}{G}$$

where:

x_t = distance of turning point from P.V.C. or P.V.T.
g = lesser slope (ignoring signs)
L = length of curve in stations
G = algebraic difference of slopes

For the curve we are calculating, the computations would be:

$$x_t = \frac{gL}{G} = \frac{7(4)}{16} = 1.75$$

Therefore, the turning point is 1.75
175 feet, from the P.V.T. (station 30

The vertical offset for the tur
would be found by the formula:

$$y_t = \left(\frac{x_t}{l}\right)^2 e$$

For this curve the computations woul

$$y_t = \left(\frac{x_t}{l}\right)^2 e = \left(\frac{1.75}{2}\right)^2 8 = 6$$

The elevation of the P.O.V.T. at 3C
be 237.37, calculated as explained
elevation on the curve would be 237
= 231.25.

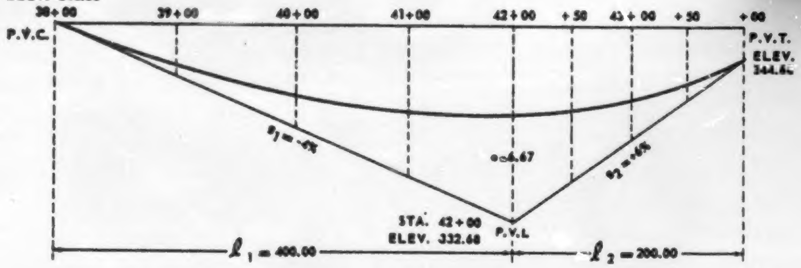
STEP 8: Check your work.

One of the characteristics of a s
parabolic curve is that the second
between successive grade elevations
tions are constant. In computing th
second differences (columns 7 and 8
take the plus or minus signs into co
When you round off your grade eleva
according to the degree of precisio
you introduce an error which will
second differences to vary slightly
another. However, the slight variatic
detract from the value of the second
as a check on your computations
cautioned that the second differenc
always come out EXACTLY even a
is merely a coincidence that the se
ences have come out exactly the s
particular problem.

Unsymmetrical Vertical Curves

An UNSYMMETRICAL VERTIC
is a curve in which the horizontal di
the P.V.I. to the P.V.C. is differer
horizontal distance between the P.
P.V.T. In other words, l_1 does NO
Unsymmetrical curves are sometime
as having unequal tangents and are r
"dog legs."

Figure 8-43 shows an unsymme
with a horizontal distance of 400 fee



45.371

Figure 8-43.—Unsymmetrical vertical curve.

Third, the turning point is not necessarily above or below the tangent with the lesser slope. The horizontal location is found by one of two formulas:

from the P.V.C.

$$x_t = \frac{(l_1)^2\, g_1}{2e}$$

from the P.V.T.

$$x_t = \frac{(l_2)^2\, g_2}{2e}$$

The procedure is to estimate which side of the P.V.I. the turning point is on, then use the proper formula to find its location. If the

Col. 1 Stations	Col. 2 Elevations on tangent	Col. 3 x/l	Col. 4 $(x/l)^2$	Col. 5 Vertical offsets	Col. 6 Grade elevation on curve	
38+00 (*P. V. C.*)	348. 68	0	0	0	348. 68	100 foot stations.
39+00 ($g_1 = -4\%$)	344. 68	¼	1/16	+ . 42	345. 10	
40+00	340. 68	½	¼	+ 1. 67	342. 35	
41+00	336. 68	¾	9/16	+ 3. 75	340. 43	
42+00 (*P. V. I.*)	332. 68	1	1	+ 6. 67	339. 35	50 foot stations.
42+50 ($g_2 = +6\%$)	335. 68	¾	9/16	+ 3. 75	339. 43	
43+00	338. 68	½	¼	+ 1. 67	340. 35	
43+50	341. 68	¼	1/16	+ . 42	342. 10	
44+00 (*P. V. T.*)	344. 68	0	0	0	344. 68	

45.372

Figure 8-44.—Table of computations of elevations on an unsymmetrical vertical curve.

ula indicates that the turning point is on pposite side of the P.V.I., you must use the formula to determine the correct location. example, you estimate that the turning is between the P.V.C. and P.V.I. for the in figure 8-43. Solving the formula:

curve in figure 8-43. Solving the formula:

$$x_t = \frac{(l_1)^2\, g_1}{2e} = \frac{(4)^2\; 4}{(2)\, 6.67} = 4.80 \text{ or}$$

station 42+80.

Station 42+80 is between the P.V.I. and P.V.T., so use the formula:

$$x_t = \frac{(l_2)^2}{2e}\, g_2 = \frac{(2)^2\; 6}{(2)\, 6.67} = 1.80 \text{ or}$$

station 42+20.

Station 42+20 is the correct location of the turning point. The elevation of the P.O.V.T., the amount of the offset, and the elevation on the curve is determined as previously explained.

Checking the Computation by Plotting

Always check your work by plotting the grade tangents and the curve in profile on an exaggerated vertical scale; that is, with the vertical scale perhaps 10 times the horizontal scale. The details of profile plotting are covered in *Engineering Aid 3 & 2*. After the P.O.V.C.'s have been plotted, you should be able to draw a smooth parabolic curve through the points with the help of a ship's curve or other appropriate irregular curve; if you can't, check your computations.

Using a Profile Work Sheet

After you have had some experience computing curves using a table as shown in the foregoing examples, you may wish to eliminate the table and write your computations directly on a work print of the profile. The engineer will set the grades and indicate the length of the vertical curves. You may then scale the P.V.I. elevations and compute the grades if the engineer hasn't done so. Then, using the calculating machine, compute the P.O.V.T. elevations at the selected stations. You will find that you can set up the computations in the calculating machine so that you can carry the grades, the stations, and the elevations in the machine from one end of the profile to the other, checking in at each previously set P.V.I. elevation. Write the tangent

previously set P.V.I. elevation. Write the tangent

elevation at each station on the worksheet. Next, compute e; in the many cases where three significant figures are sufficient, it is most convenient to use your slide rule. Then compute each vertical offset: mentally note the x/l ratio, then square it and multiply by e on your slide rule. Write the offset on the work print opposite the tangent elevation. Next, add or subtract the offsets from the tangent elevations (either mentally or on the machine) to give you the curve elevations which you then record on the work sheet. Plot the P.O.V.C. elevations, and draw in the curve. Last, put the necessary information on the original tracing. The information generally shown includes grades, finished elevations, length of curve, location of P.V.C., P.V.I., P.V.T., and the e. Figure 8-45 shows a portion of a typical work sheet completed up to the point of drawing the curve.

FIELD STAKEOUT OF VERTICAL CURVES

The stakeout of a vertical curve consists, basically, of marking the finished elevations in the field to guide the construction personnel. Detailed procedures for setting grade stakes are covered in *Engineering Aid 3 & 2*. The procedure for setting a grade stake is the same whether it's on a tangent or on a curve, so a vertical curve introduces no special problem. As indicated before, stakes are sometimes set closer together on a curve than on a tangent; but this will usually have been foreseen, and the plans will show the finished grade elevations at the required stations. If, however, the field conditions do require a stake at an odd plus on a curve, you may compute the needed P.O.V.C. elevation in the field using the data given on the plans and the computational procedures in this chapter.

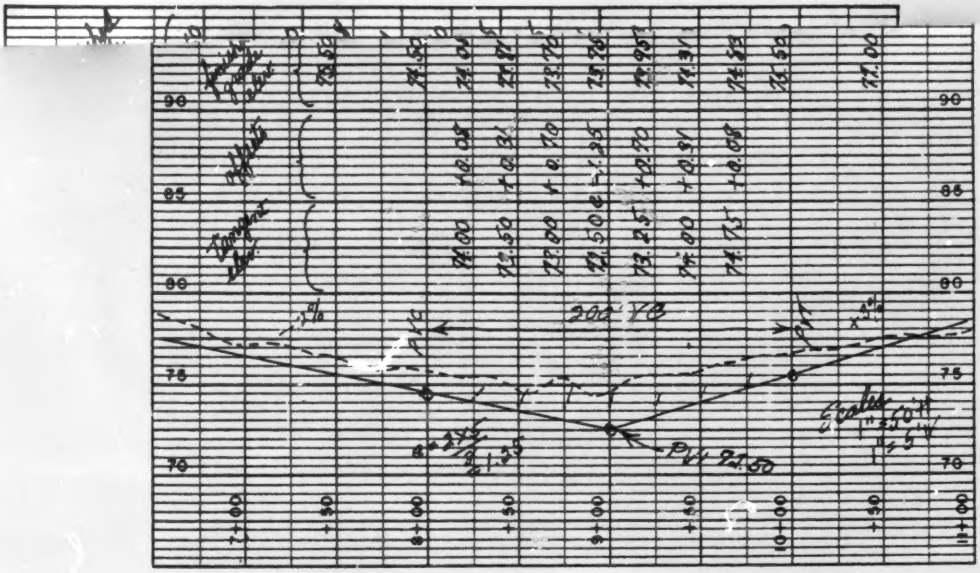


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Figure 8-45.—Profile work sheet.